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AND
CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

JANUARY, 1952. **Vol. XLVII, No. 1**

FORTY-SEVENTH YEAR OF PUBLICATION

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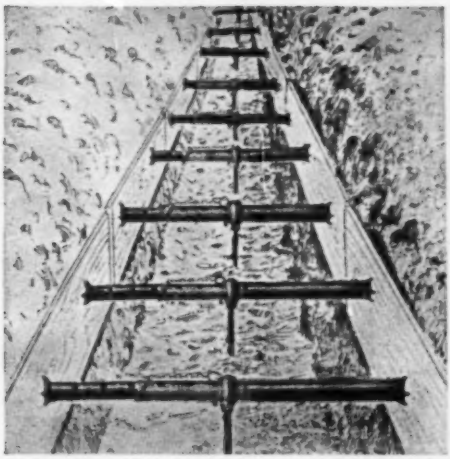

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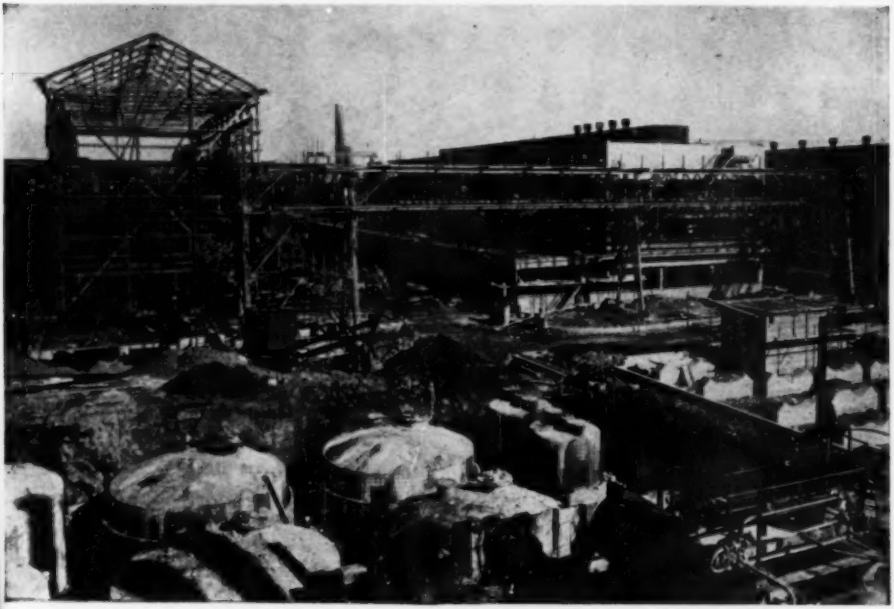

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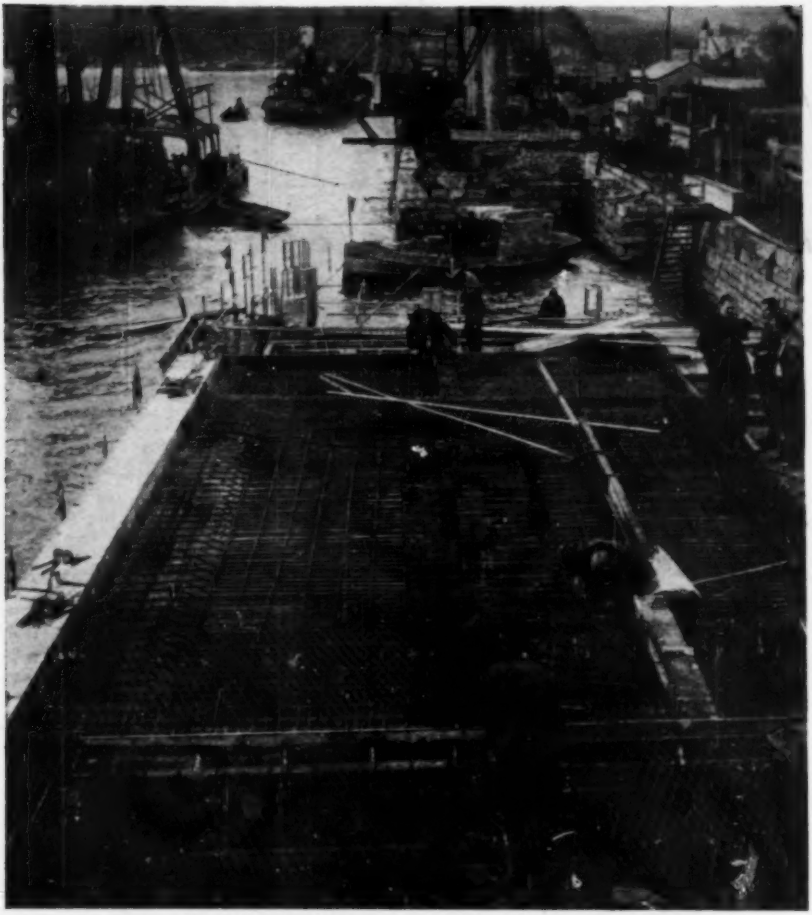

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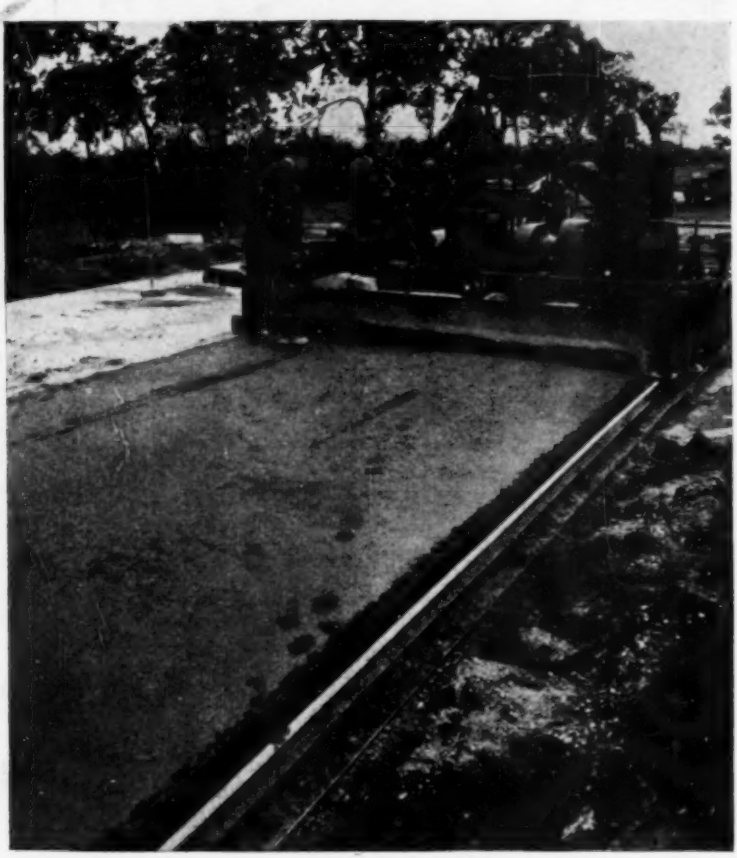

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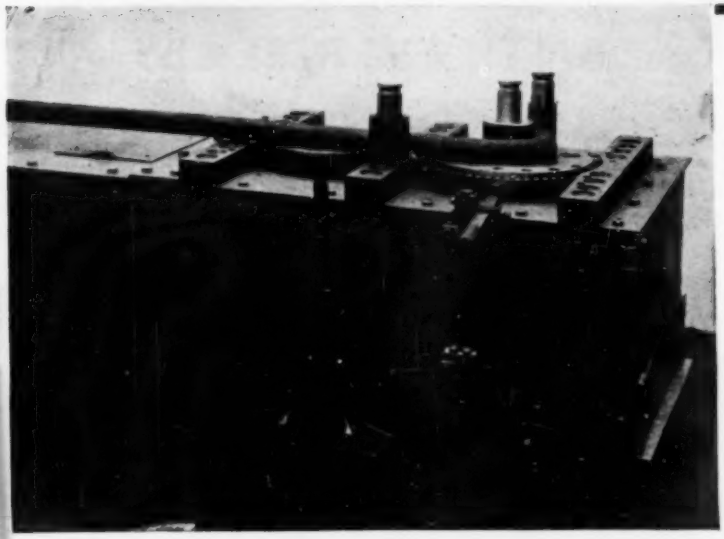

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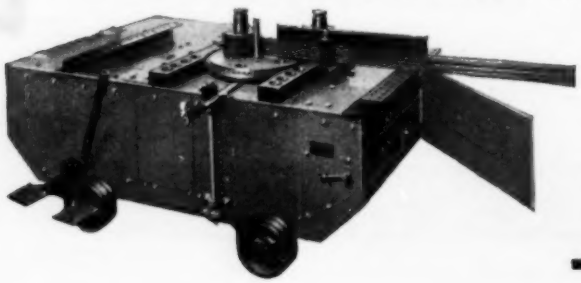

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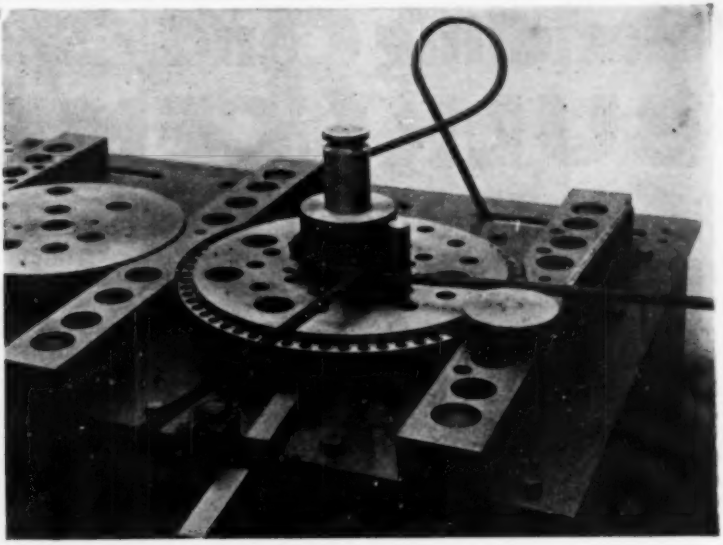

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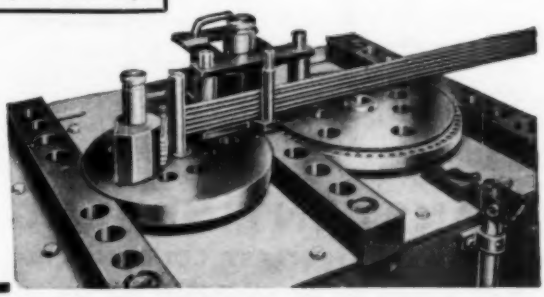

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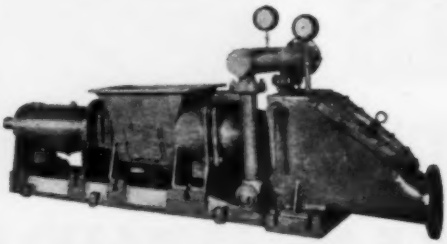

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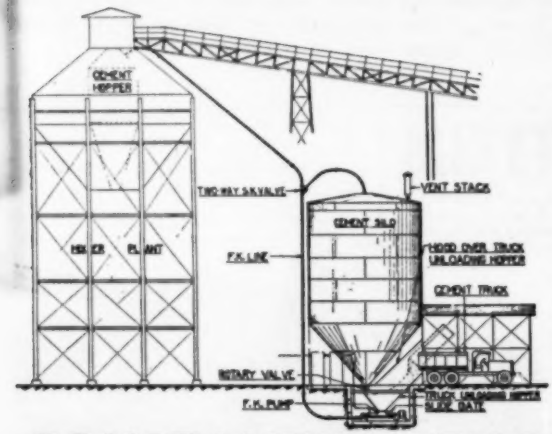

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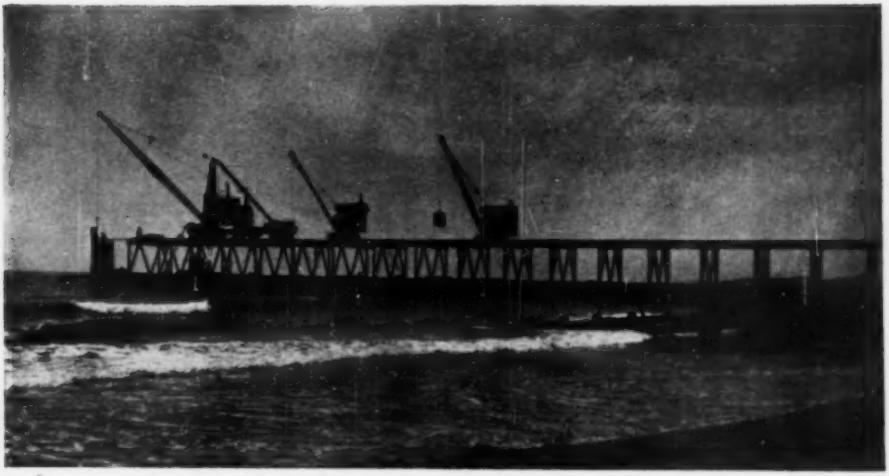

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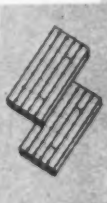

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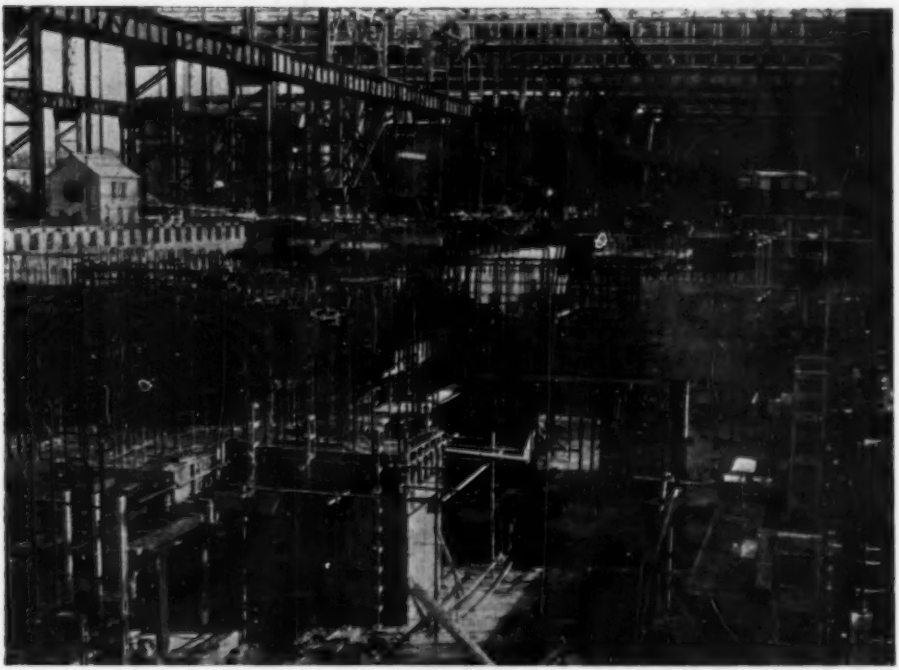

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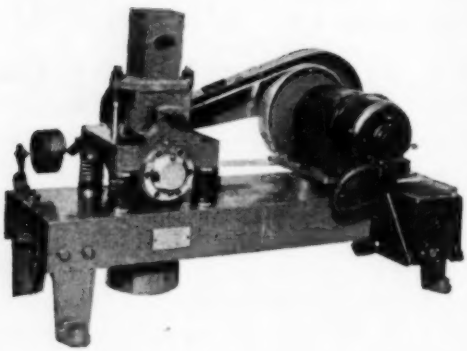

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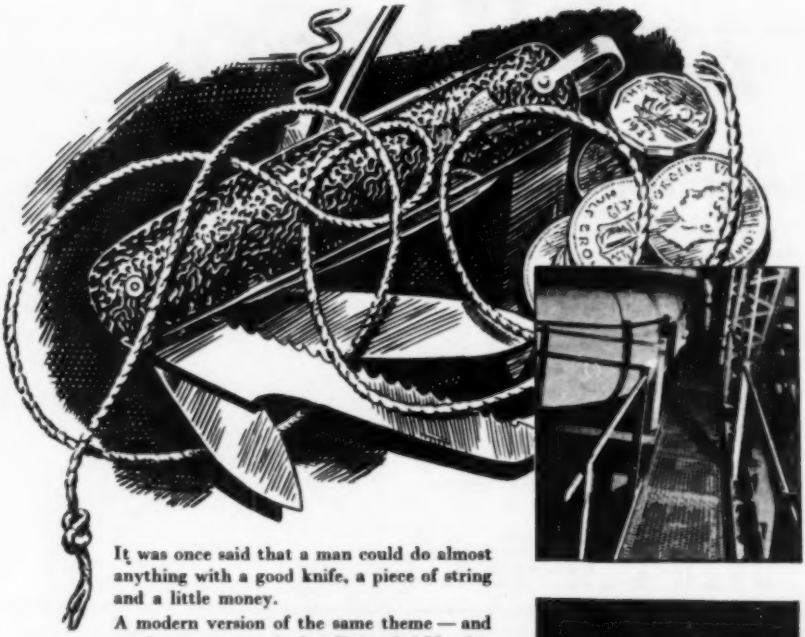

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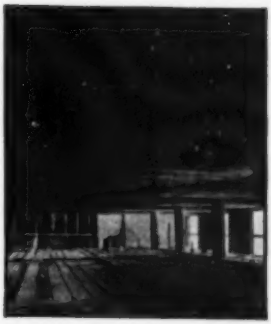

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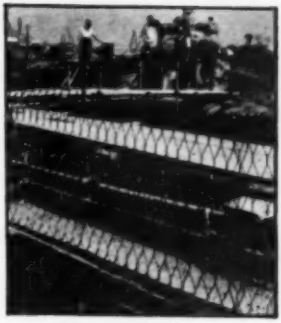

● *Photographs illustrate (Top) Expanded Metal walkways in the Royal Festival Hall. (Centre) "BB" Lathing ceiling at Bow Road Methodist Church, London. Architects: Messrs. Mauger & May, F/F.R.I.B.A. (Bottom) Reinforcement for concrete—Electricity Sub-station, Brentford, for the Southern Electricity Board.*

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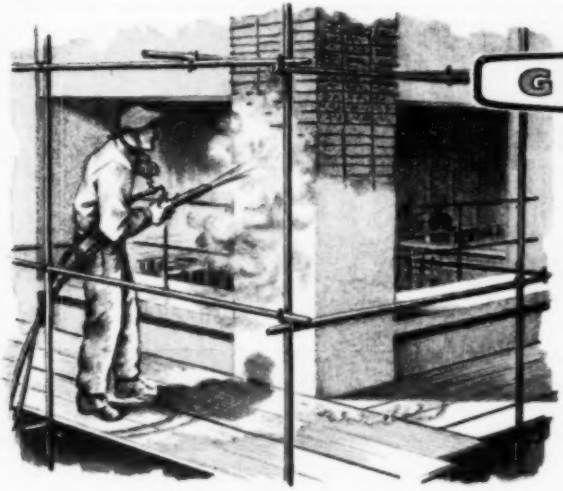

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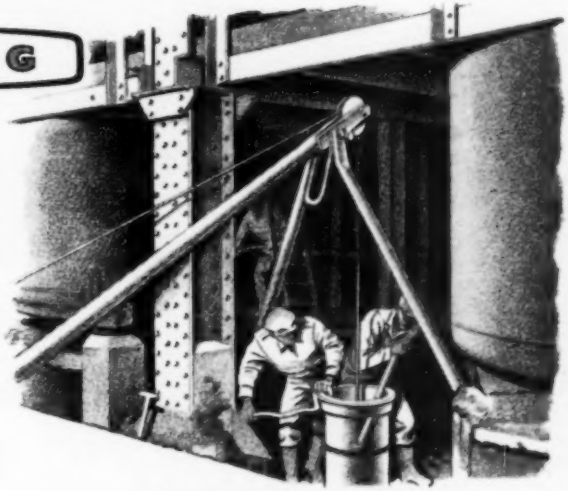

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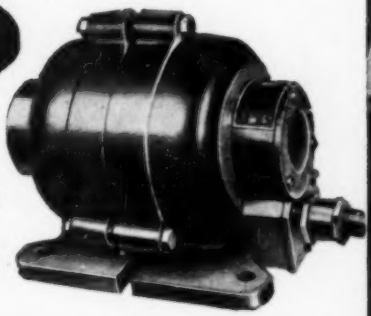

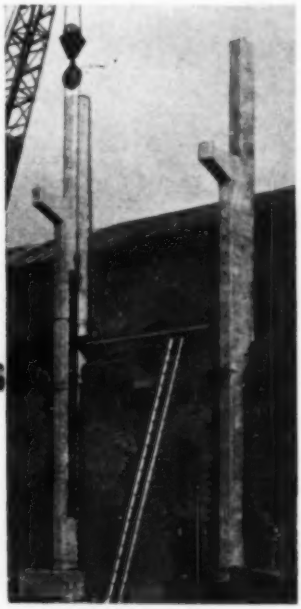

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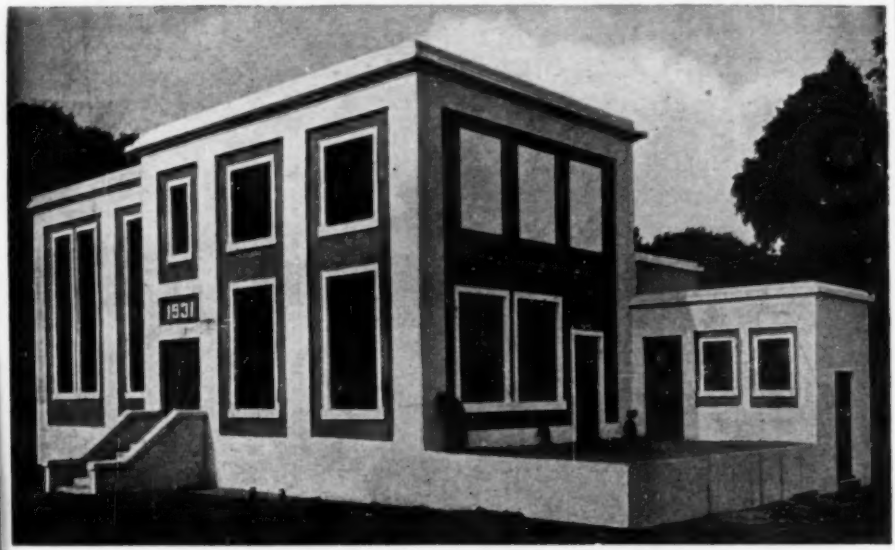

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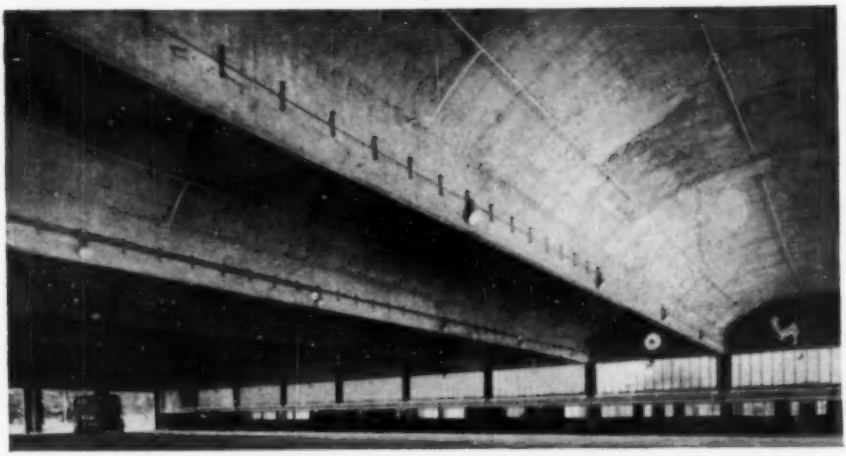

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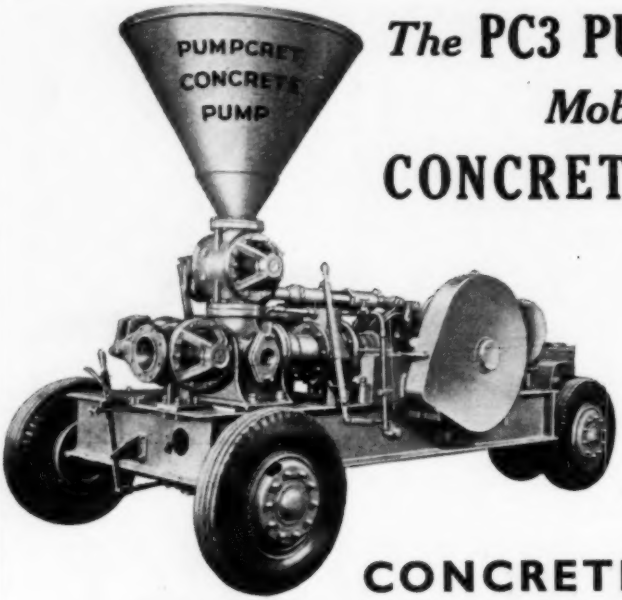

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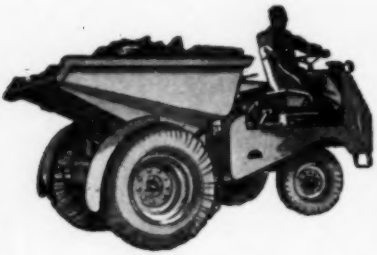

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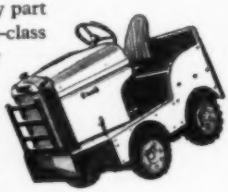

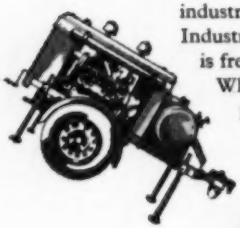

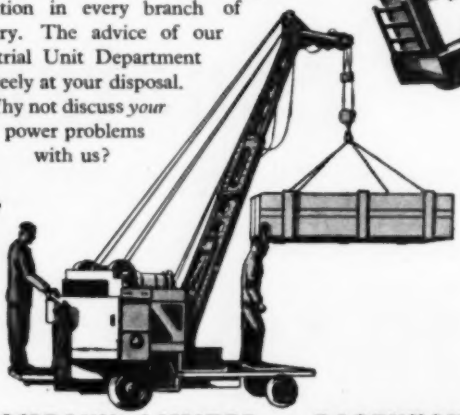

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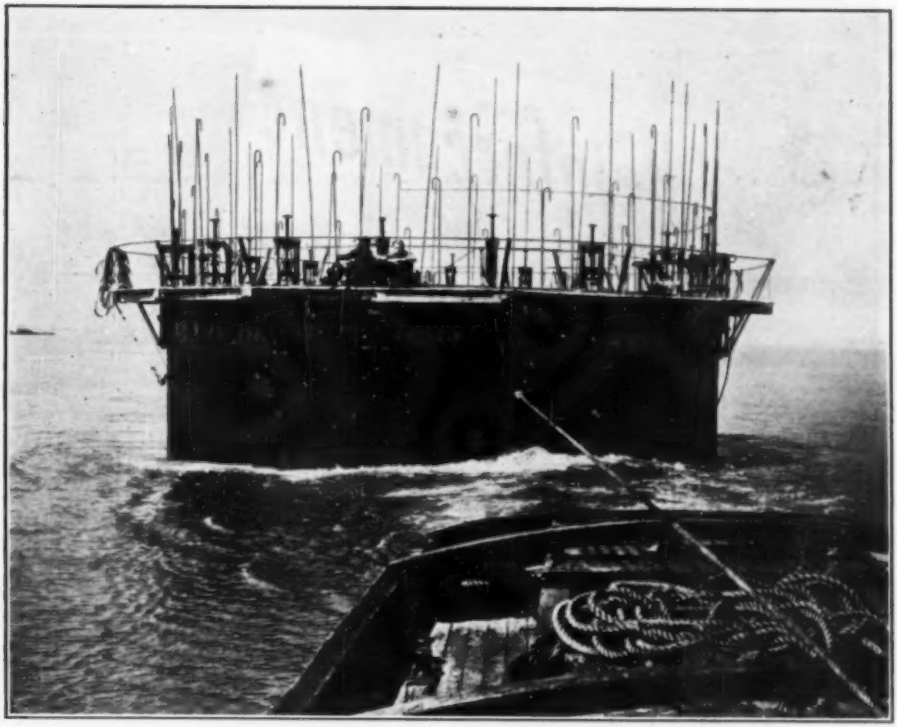

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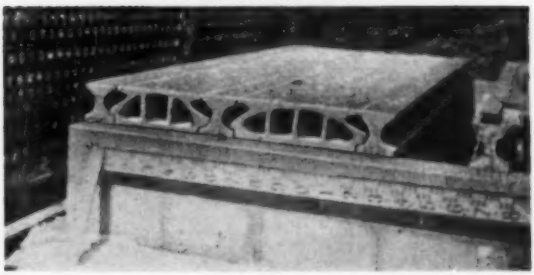

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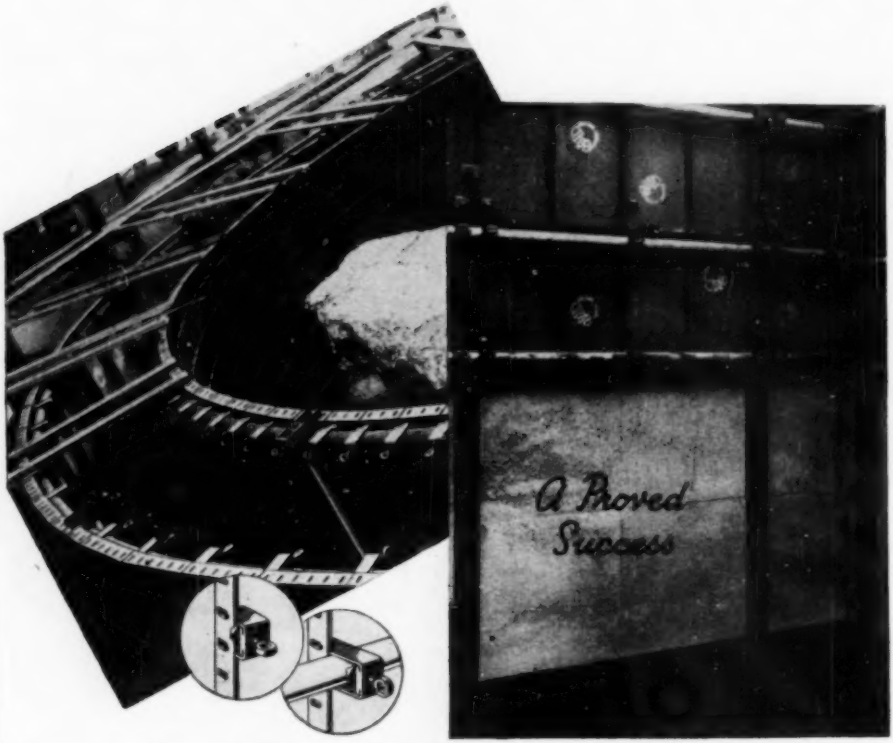

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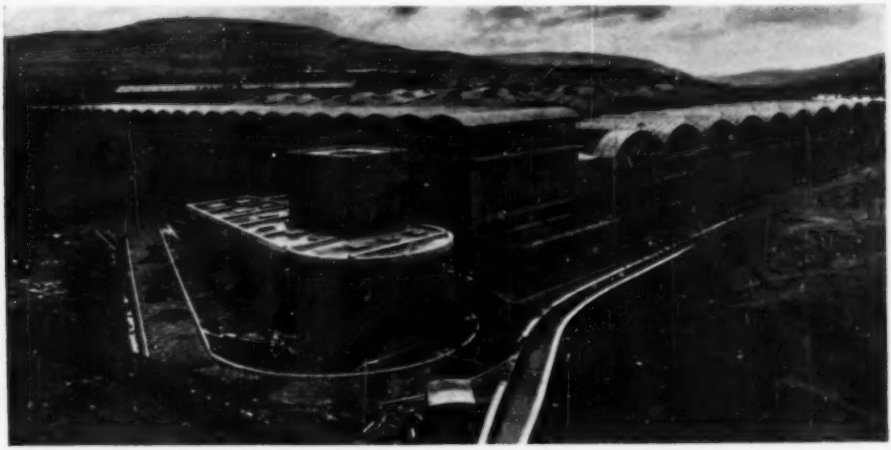

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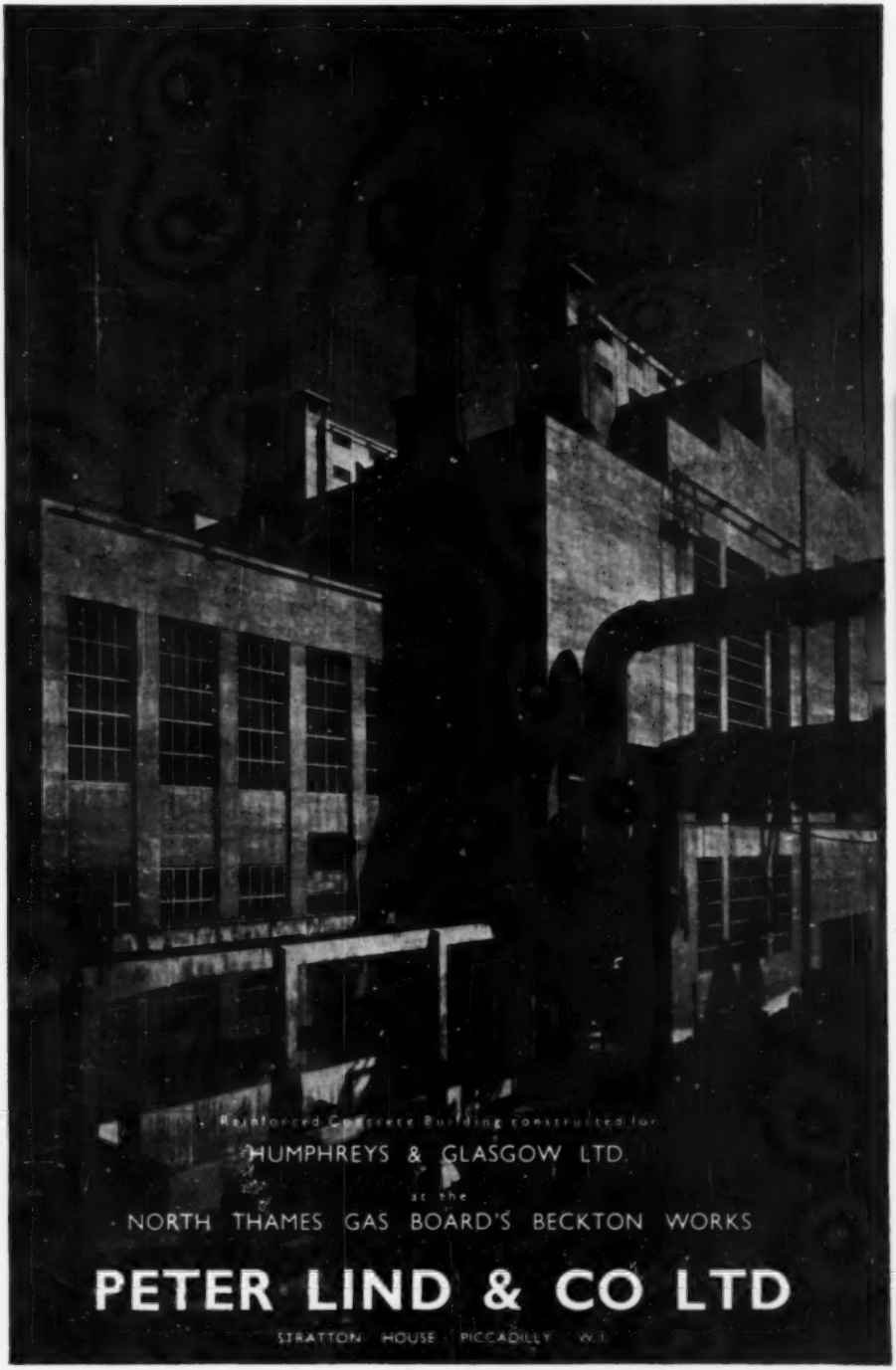
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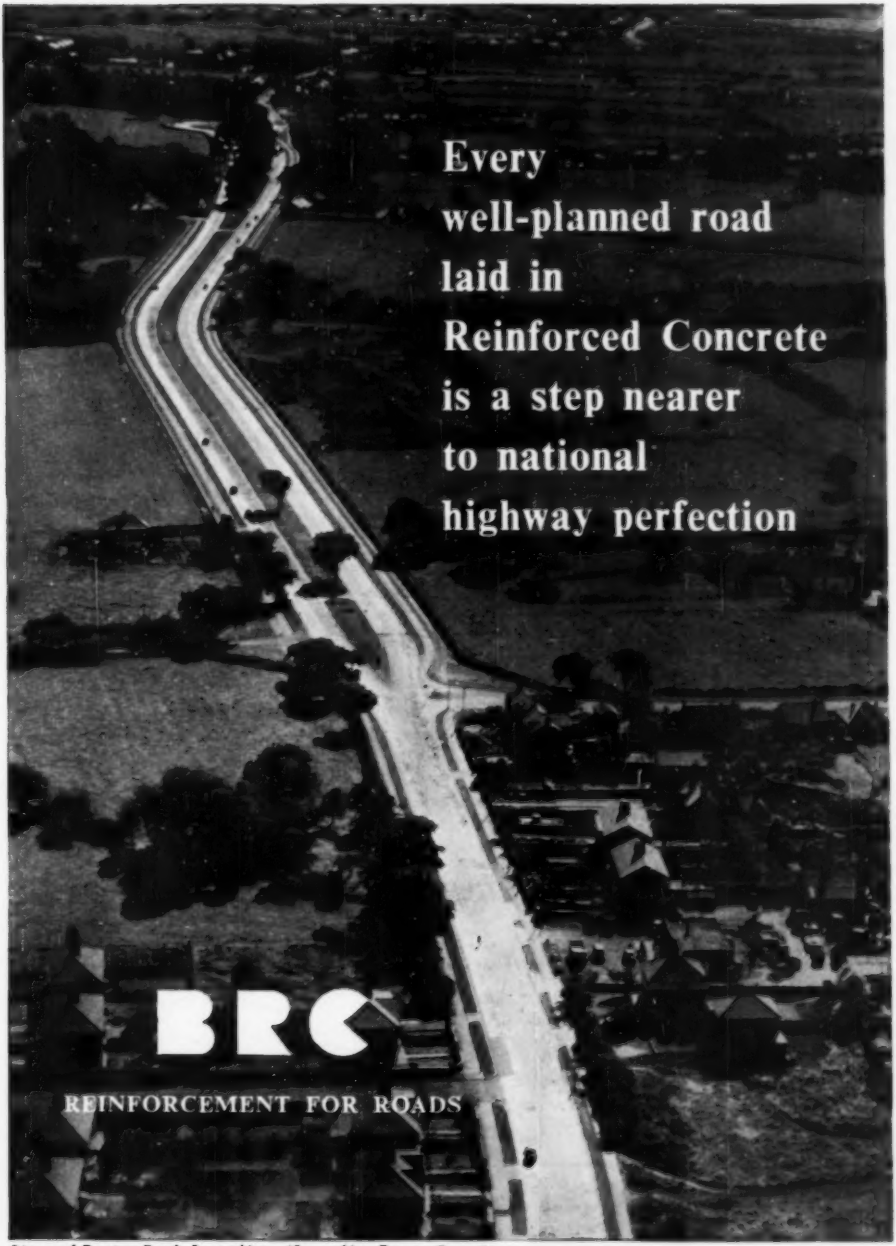

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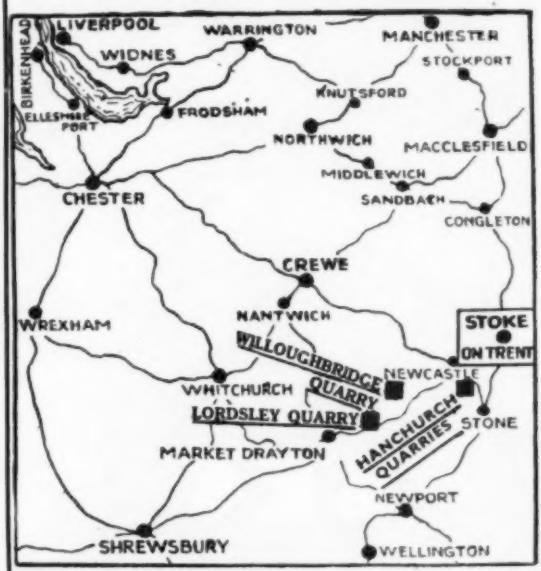

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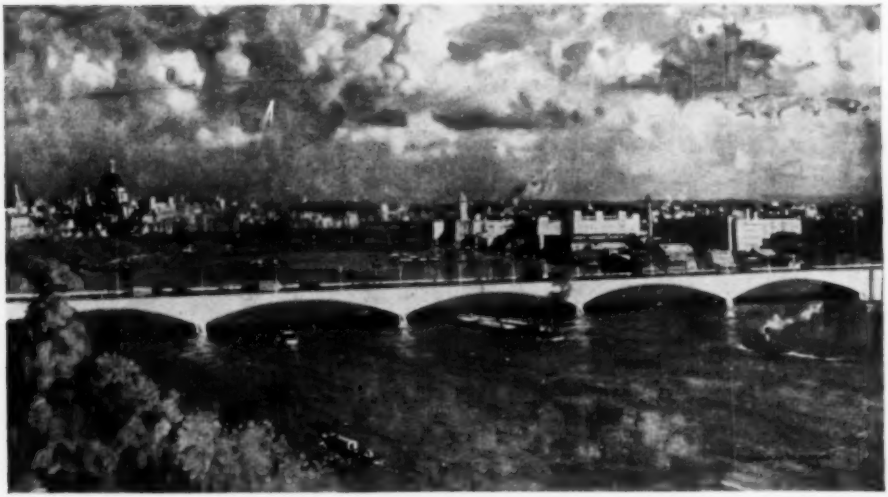

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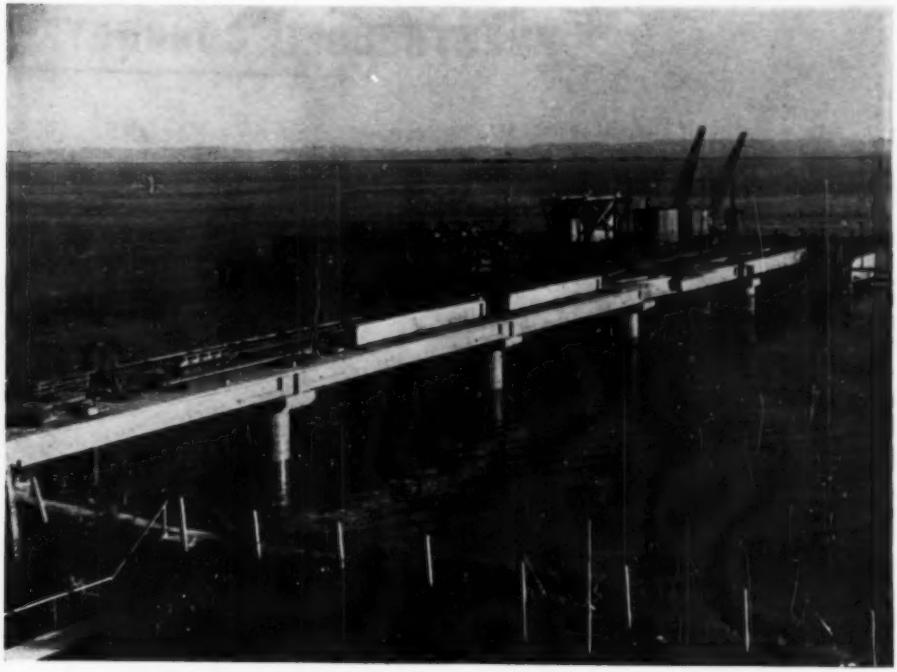

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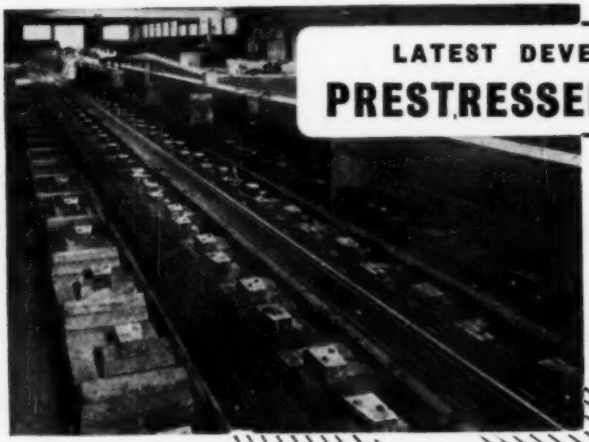

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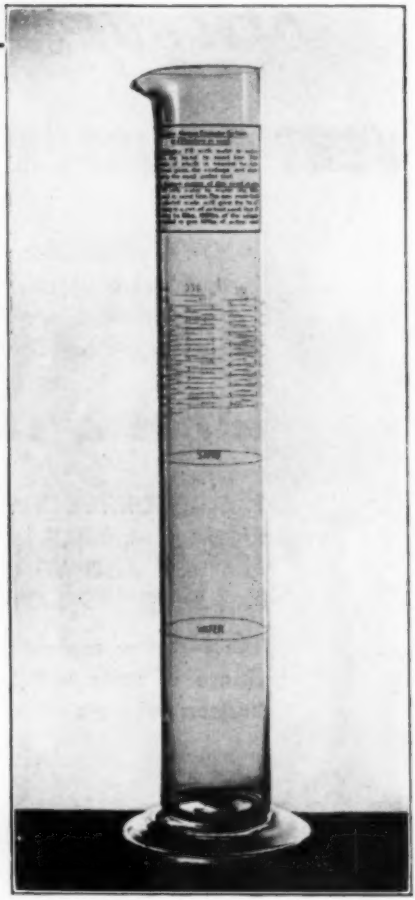

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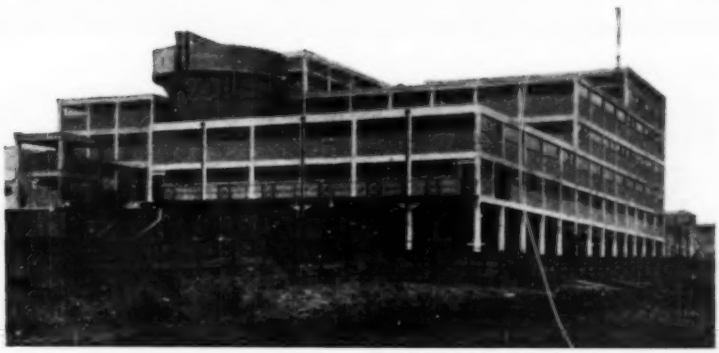

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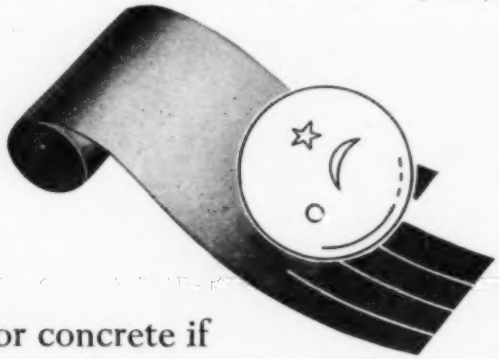

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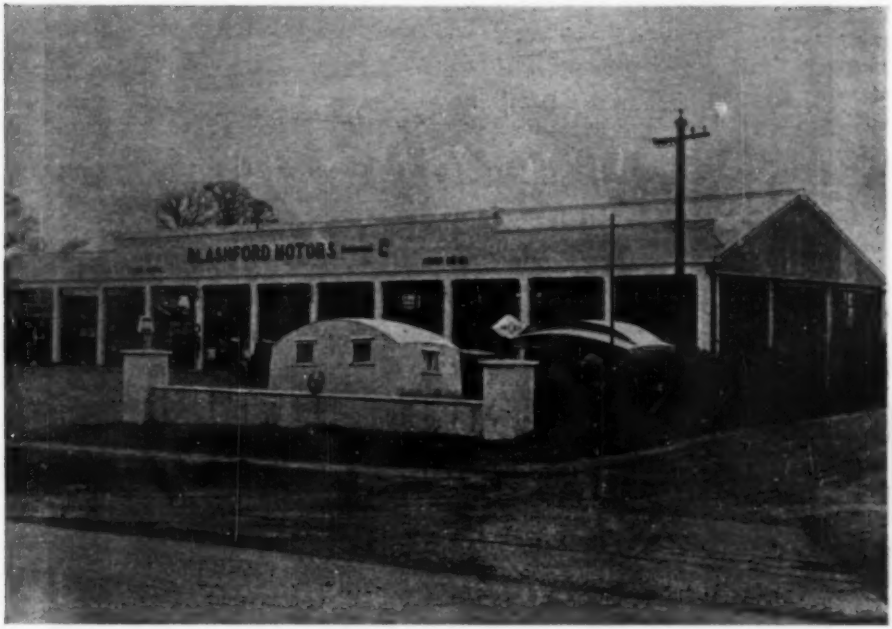

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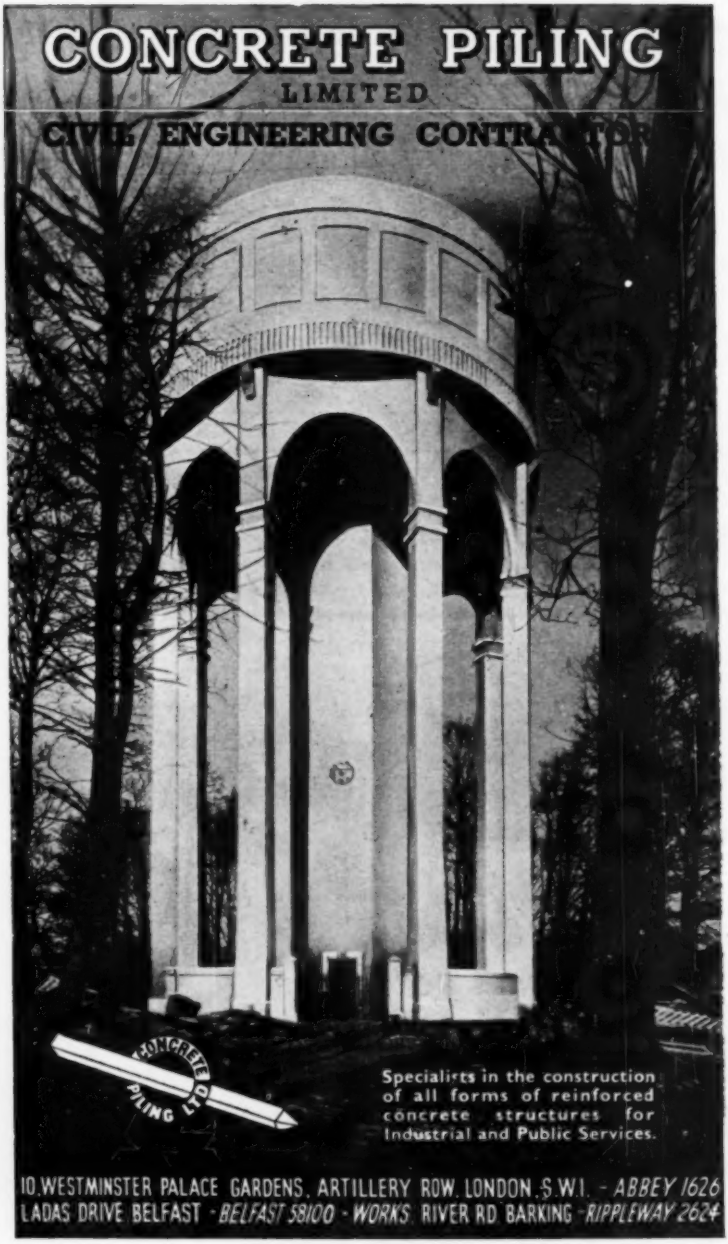
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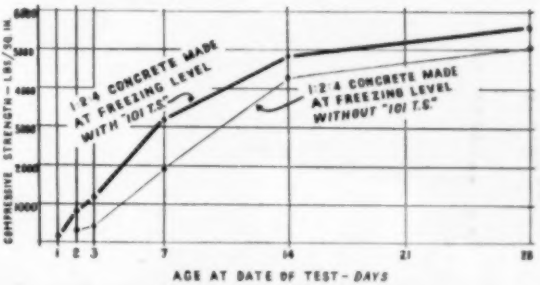
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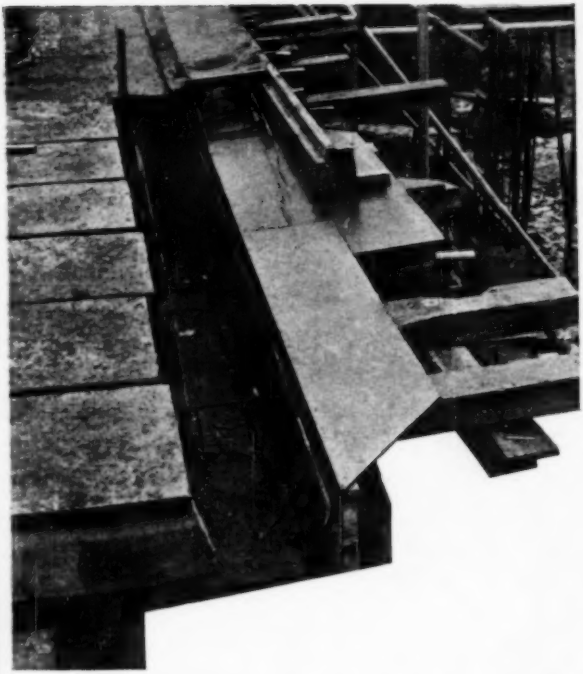

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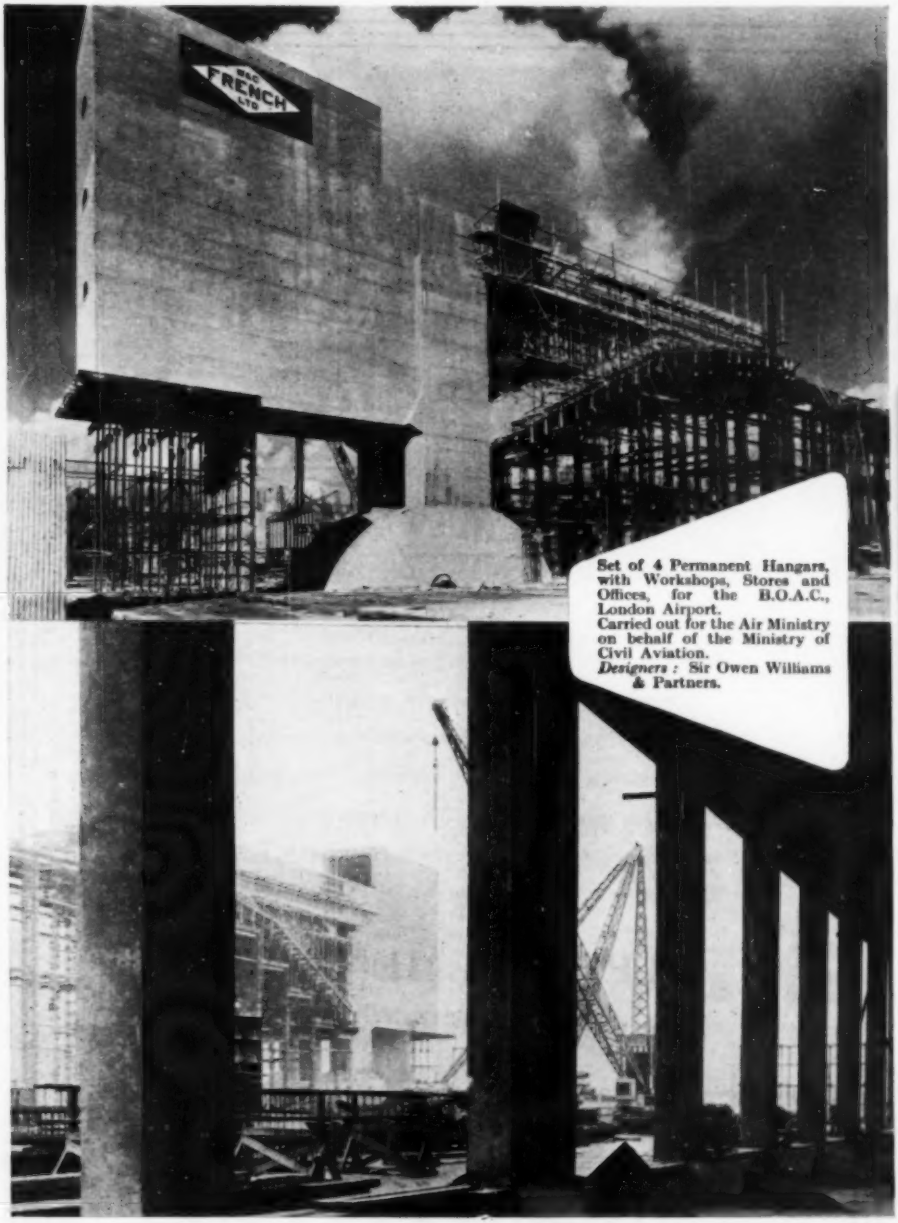

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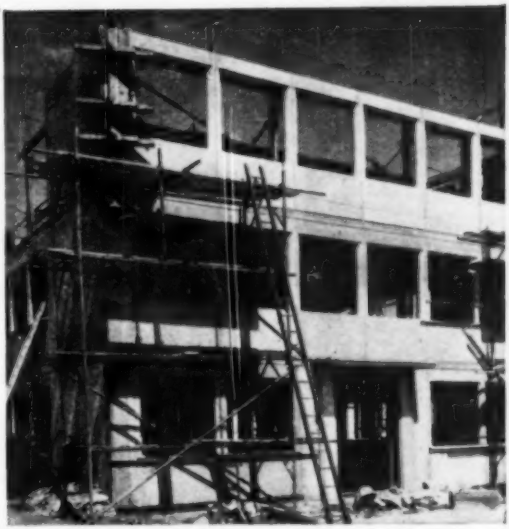

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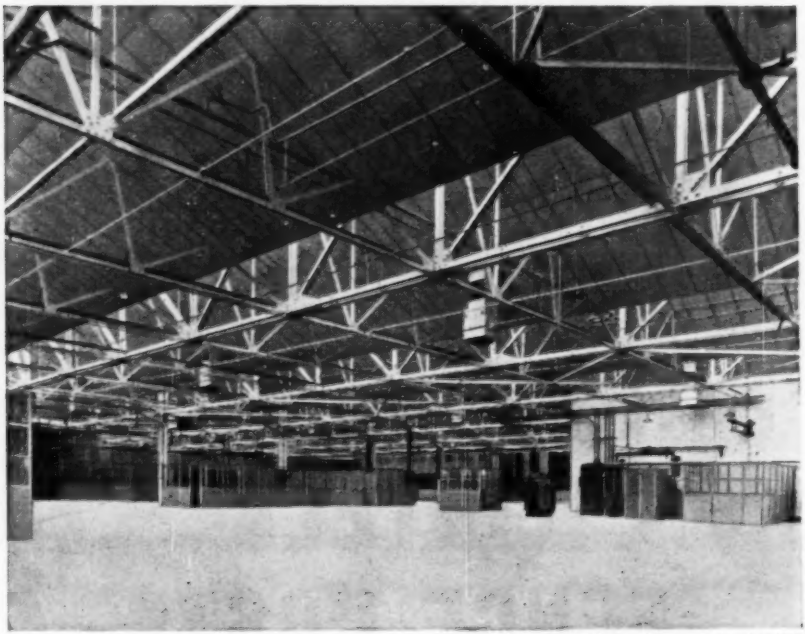

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Volume XLVII. No. 1. LONDON, JANUARY, 1952.

EDITORIAL NOTES

Methods of Structural Analysis.

THE objects of structural analysis are to calculate with reasonable accuracy the magnitude of the forces and bending moments produced by the loads, and to obtain a mental picture of how a loaded structure deforms. The results of computations, however, cannot be more accurate than the assumptions on which they are based. The analysis of a structure assumed to have pin joints is relatively simple, but the assumption that the connections are pin joints may be far from true. The behaviour of a structure with rigid joints, that is when angular deformation of one member causes similar deformation of other members meeting at each joint, is more complex and the assumptions may be less accurate because not only are the laws of statics concerned but the dimensions of the members and the elastic and other properties of the material have to be considered; for reinforced concrete these properties are not generally known precisely. Thus the analysis of a statically-indeterminate structure is more complex, and the method adopted can vary from an approximate estimate of the forces and moments (which may be fairly accurate if done by an experienced designer) to elaborate " rigid " analyses which may be merely an exhibition of the designer's skill as a mathematician rather than as an engineer. Intermediately there are several methods which are of practical application.

Because of the complexity of the problem many methods have been produced having as their object the simplification of the calculations without impairing greatly the accuracy. In the nineteenth century the fundamental theories were developed and to some extent less complex practical methods were evolved. More recently methods such as moment-distribution (and other methods of successive approximation) and column analogy have been devised, and these appeal to designers since less laborious mathematics or arithmetic is required. The choice of method is generally based on two criteria, one being the aptitude of the designer to understand and to be able to apply some methods more readily than others, and the other being that some methods are more easily applied to certain problems than are others. For example, the principles of area-moments are fundamental but are most readily applied to statically-indeterminate beams, whereas analysis by slope-deflection methods, which are based thereon, is applicable to most frames but great care with regard to algebraical signs is necessary.

The method of strain-energy and the related method of least work are applicable to all frames and in a modified form to arches, but the integrations must be carefully made. When structures are subjected to moving loads some of these methods are tedious.

The requirement of a busy designer is a method that is easily memorised as regards procedure and signs, that adopts the same routine for all types of structures (so that the chances of error in procedure are reduced), that requires the use of a slide-rule only, and that is so far as possible self-checking. Such a method is the Displacement Method, which was described in this journal for March, 1951, and an example of the application of which is given in this number. The method, which is applicable to simple and complex frames, closed frames, and continuous beams, whether the members are of constant or non-constant section, is not a new one, but has been used in practical design for upwards of fifteen years.

Briefly the Displacement Method consists of assuming each joint of the frame to be rotated in turn by an arbitrary amount in an anticlockwise direction and tabulating the effects on the members meeting at each joint. If the structure is subjected to sway, an arbitrary lateral displacement from left to right is also assumed and the effect on the members concerned is likewise entered in the tabulation. The effects, that is the bending moments and shearing forces, are expressed in terms of the angle of rotation (or the displacement) and the dimensions of the member, and are determined by the application of a few easily remembered rules. The bending moments produced by the loads at the ends of each loaded member, assuming the ends to be fixed, are also entered in the tabulation. At this stage the tabulation can be readily checked for clerical errors since in each row and column certain terms, and these terms only, should appear. Succeeding steps are merely algebraical. Consideration of the equilibrium of each joint gives the same number of equations as there are unknown rotations or displacements. The equations are simply the summation of the terms in appropriate columns of the tabulation. The solution of the simultaneous equations to give the expressions for the rotations and displacements presents no difficulty in the case of frames having three or four joints and, if a routine method of solution is applied, need not do so in the case of more complex frames. From the tabulations the expressions for each bending moment and shearing force can also be written down, and the magnitudes of these actions are obtained immediately when the rotations and displacements are substituted in the equations. Arbitrary deformation diagrams are used throughout, and these help to prevent inaccuracies due to guessing wrongly the shape of the deformed structure.

Some designers can, from experience, obtain the critical bending moments and forces very closely by approximations of the $\frac{wl^2}{10}$-type, but these methods may be unreliable if the structure is subjected to sway. By the displacement method, an exact solution (exact, that is, within the degree of accuracy of the assumptions made in all elastic analyses) is obtained almost directly by the routine application of simple rules. It is true that simple frames subjected to commonly occurring loads can be most easily designed by substitution in formulæ which are given in many books on structural design, but for unusual frames or uncommon loads familiarity with a practical and accurate method of analysis is important.

Analysis of Groups of Piles by the Displacement Method.

By G. P. MANNING, M.Eng., M.Inst.C.E.

ALTHOUGH piled foundations are not usually considered as frames, the distribution of the load from a foundation or abutment to the piles may be calculated by the displacement method, the principles of which were described in this journal for March, 1951.

A Pair of Inclined Piles.

Fig. 1*a* shows the simplest case of two identical raking piles subjected to a horizontal pull *H*. The piles are usually designed as pin-jointed members as in *Fig.* 1*b*, but they are generally monolithic with the superstructure at B and fixed at A and C as in *Fig.* 1*c*. The piles in *Fig.* 1*b* are subjected to direct tension or compression only, but those in *Fig.* 1*c* are also subjected to bending moments.

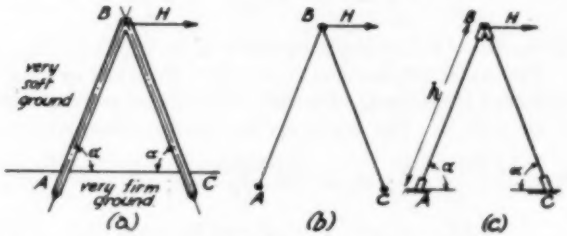


Fig. 1.

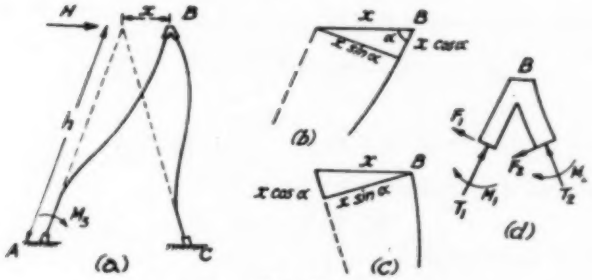


Fig. 2.

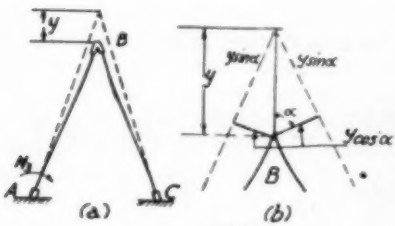


Fig. 3.

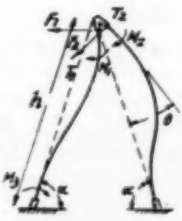


Fig. 4.

Fig. 5.

The movement of joint B due to the load H will in general be (1) horizontal displacement x (*Fig.* 2); (2) vertical displacement y (*Fig.* 3); and (3) rotational displacement θ (*Fig.* 4).

The effect on pile AB of the horizontal movement of joint B in *Fig.* 2*a* is shown to a larger scale in *Fig.* 2*b*, and the effect on pile BC is shown in *Fig.* 2*c*. The horizontal displacement x of the head of pile AB can be regarded as a displacement $x \sin \alpha$ at right angles to AB and an extension $x \cos \alpha$. Similarly the head of pile BC is displaced $x \sin \alpha$ at right-angles to BC and contracts $x \cos \alpha$. *Fig.* 2*d* shows joint B to a larger scale and indicates the actions of the members on the joint. If A is the effective cross-sectional area of the pile and K is the value of $\frac{I}{h}$, the reactions due to x are

$$M_1 = \frac{6EKx \sin \alpha}{h}; \quad F_1 = \frac{12EKx \sin \alpha}{h^2}; \quad T_1 = -\frac{AEx \cos \alpha}{h}.$$

$$M_2 = \frac{6EKx \sin \alpha}{h}; \quad F_2 = \frac{12EKx \sin \alpha}{h^2}; \quad T_2 = \frac{AEx \cos \alpha}{h}.$$

Fig. 3*a* shows the vertical displacement y of B, which in *Fig.* 3*b* is shown to a larger scale. Pile AB is displaced at right-angles from left to right by an amount $y \cos \alpha$ and shortened by $y \sin \alpha$. Pile BC is displaced $y \cos \alpha$ from right to left and shortened by $y \sin \alpha$. The reactions on joint B due to y are:

$$M_1 = \frac{6EKy \cos \alpha}{h}; \quad F_1 = \frac{12EKy \cos \alpha}{h^2}; \quad T_1 = \frac{AEy \sin \alpha}{h}.$$

$$M_2 = -\frac{6EKy \cos \alpha}{h}; \quad F_2 = -\frac{12EKy \cos \alpha}{h^2}; \quad T_2 = \frac{AEy \sin \alpha}{h}.$$

The angular displacement θ of B (*Fig.* 4) produces

$$M_1 = 4EK\theta; \quad F_1 = \frac{6EK\theta}{h}; \quad T_1 = 0.$$

$$M_2 = 4EK\theta; \quad F_2 = \frac{6EK\theta}{h}; \quad T_2 = 0.$$

For rotational equilibrium of B,

$$M_1 + M_2 = 0 \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

For vertical equilibrium,

$$F_1 \cos \alpha - F_2 \cos \alpha + T_1 \sin \alpha + T_2 \sin \alpha = 0 \quad . \quad . \quad . \quad (2)$$

For horizontal equilibrium,

$$F_1 \sin \alpha + F_2 \sin \alpha - T_1 \cos \alpha + T_2 \cos \alpha = H \quad . \quad . \quad (3)$$

From equation (1),

$$\left(\frac{6EKx \sin \alpha}{h} + \frac{6EKy \cos \alpha}{h} + 4EK\theta\right) + \left(\frac{6EKx \sin \alpha}{h} - \frac{6EKy \cos \alpha}{h} + 4EK\theta\right) = 0,$$

from which

$$\theta = -\frac{1{\cdot}5x \sin \alpha}{h}.$$

From equation (2): $$\frac{24EKy\cos^2\alpha}{h^2}+\frac{2AEy\sin^2\alpha}{h}=0,$$

from which $y=0$.

From equation (3): $$\frac{24EKx\sin^2\alpha}{h^2}+\frac{12EK\theta\sin\alpha}{h}+\frac{2AEx\cos^2\alpha}{h}=H.$$

Substituting θ in terms of x and transposing, $$Ex=\frac{H}{\frac{6K\sin^2\alpha}{h^2}+\frac{2A\cos^2\alpha}{h}}.$$

From *Figs.* 2*a*, 3*a*, and 4, $$M_3=\frac{6EKx\sin\alpha}{h}+\frac{6EKy\cos\alpha}{h}+2EK\theta.$$

Since $y=0$, $M_3=\frac{3EKx\sin\alpha}{h}$. Substituting for Ex, and if $A=d^2$ and $I=\frac{d^4}{12}$ (where d is the width of a square pile),

$$M_3=\frac{Hh\operatorname{cosec}\alpha}{2\left[1+\left(\frac{2h\cot\alpha}{d}\right)^2\right]}.$$

If $d=1{\cdot}17$ ft., $h=40$ ft., $H=15$ tons, and $\cot\alpha=0{\cdot}333$, M_3 is only 16,200 in.-lb. Since $y=0$,

$$M_1=\frac{6EKx\sin\alpha}{h}+4EK\theta, \text{ and } M_2=\frac{6EKx\sin\alpha}{h}+4EK\theta.$$

That is $M_1=M_2$, and, since $M_1+M_2=0$, $M_1=-M_2=0$.

Similarly, if the pair of piles carries a vertical load V as in *Fig.* 5 with displacements as in *Fig.* 3*b*,

$$M_1=M_3=-M_2=\frac{3Vh\sec\alpha}{12+\frac{Ah\sin^2\alpha}{K\cos^2\alpha}}.$$

Substituting, $A=d^2$ and $K=\frac{d^4}{12h}$,

$$M_1=M_3=\frac{Vh\sec\alpha}{4\left[1+\left(\frac{h\tan\alpha}{d}\right)^2\right]}.$$

If $d=1{\cdot}17$ ft., $h=40$ ft., $\tan\alpha=3$, and $V=60$ tons, M_1 is only 4,800 in.-lb. If the effect of the horizontal load H of 15 tons is added, the total value of M_3 is only 21,000 in.-lb., that is 14-in. square piles 40 ft. long under this condition can be assumed without appreciable error to be pin-jointed as in *Fig.* 1*b*.

Fig. 1*a* shows, however, the ideal ground for piling. Sometimes the conditions may be as in *Fig.* 6. The effective length of the pile in *Fig.* 6*a* as regards direct thrust and lateral support may be only about one-half the actual length. *Fig.* 6*b* shows conditions requiring much hard driving or jetting to penetrate the stratum of compact sand, and the effective length of the pile as regards direct thrust may be equal to the actual length, while the effective length as regards lateral support

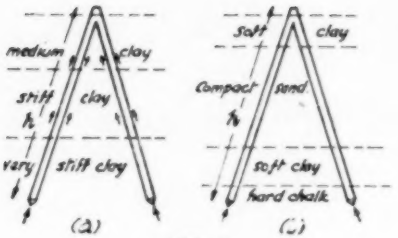


Fig. 6.

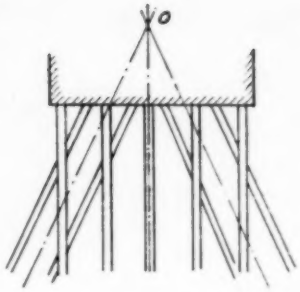


Fig. 7.

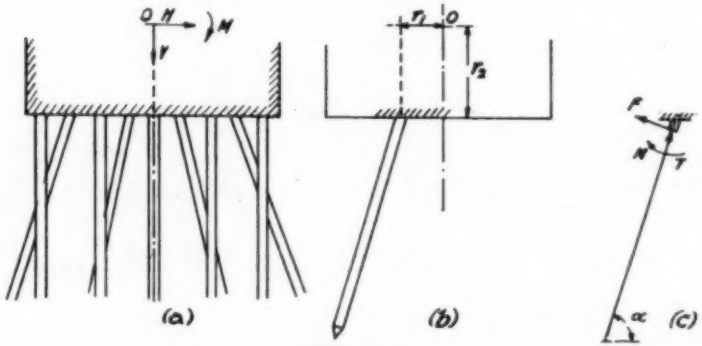


Fig. 8.

may be only one-quarter of the actual length. Consider a pair of 14-in. square piles each 40 ft. long in ground as in *Fig.* 6*b*. Assume that the direct thrust on the piles is resisted by end bearing so that the effective length of the pile for consideration of shortening under direct load is 40 ft. Assume also that the piles are held laterally by the compact sand 10 ft. below the surface of the ground so that the effective length in bending is only 10 ft. Apply vertical and horizontal loads of 60 tons and 15 tons respectively as before, with $\tan \alpha = 3$. Adjusting the primary equations for the two values of the effective length h gives

$$M_1 \text{ due to } x = \frac{6EKx \sin \alpha}{10 \text{ ft.}}; \quad T_1 \text{ due to } x = -\frac{AEx \cos \alpha}{40 \text{ ft.}}; \text{ etc.}$$

The bending moments calculated for these conditions are

$$M_1 = M_2 = +\ 0{\cdot}0474V = 76{,}500 \text{ in.-lb.}$$
$$M_3 \text{ (10 ft. below B)} = +\ 0{\cdot}0474V + 0{\cdot}575H = 309{,}500 \text{ in.-lb.}$$

Thus M_3 is nearly fifteen times the corresponding moment on similar piles driven through very soft ground. The conditions assumed in *Fig.* 6*b* are severe, but the calculations show that a hard layer near ground level may be a source of danger to a pile subjected to a horizontal load.

Fig. 8*a* shows the base of an abutment or pier on piles. It is assumed that the abutment or pier is so rigid that although it may move bodily it will not be

distorted. The total load on the piles may be expressed as a vertical load V, a horizontal thrust H, and a moment M acting about any point O. The displacements of the abutment or pier due to these loads are a horizontal movement x to the left, a vertically downward movement y, and a rotation θ about O. Consider any pile as in *Fig.* 8*b*. The horizontal displacement of the head is $x + r_2\theta$, the vertical displacement is $y + r_1\theta$, and the anticlockwise rotation is θ. The displacements cause actions M, F, and T as in *Fig.* 8*c*, the values of each being as given in the analysis relating to *Figs.* 2 to 4. Resolve the actions horizontally and vertically and compute their moments about O. Considering each pile separately and, tabulating and summating the results, we obtain the total horizontal reaction and the total vertical reaction of the piles on the abutment or pier, and the total moment of these reactions about O. The total reactions and moment must be equal and opposite to H, V, and M (in *Fig.* 8*a*) respectively.

For long piles driven through soft ground we may ignore M and F in *Fig.* 8*c*, and consider the piles as pin-jointed struts as in *Fig.* 1*b*, but if there is a hard layer of ground close to the underside of the abutment or pier we must include M and F in the analysis both as regards the stability of the abutment or pier and the stresses in the piles. In many cases the inclined piles are all driven to the same angle as in *Fig.* 7, and may be regarded as in three groups under a pier or two groups under an abutment, namely vertical piles, piles inclined to the right, and piles inclined to the left. The centre-lines of each group may have a common point of intersection which can be chosen as the point O in *Fig.* 8*a*. Each group is then treated as a unit instead of considering each pile separately.

Stresses in Steel Reinforcement.

We have received the following statement from the Reinforced Concrete Association.

In view of the urgent need for economy in the use of steel, the Council of the Reinforced Concrete Association has given renewed consideration to a suggestion that the working stress of 18,000 lb. per square inch normally permitted in mild steel reinforcement, complying with B.S. No. 785 (Rolled Steel Bars for Concrete Reinforcement), might safely be increased and the quantity used in reinforced concrete structures thus reduced.

Although no yield stress is specified in B.S. No. 785, recent tests indicate that the yield stress in mild steel bars of the smaller diameters is generally at least 36,000 lb. per square inch. This being so, the Council is satisfied that a working stress of 20,000 lb. per square inch may safely be adopted in bars of $\frac{1}{2}$-in. diameter and less, except in the design of liquid-retaining and other special structures.

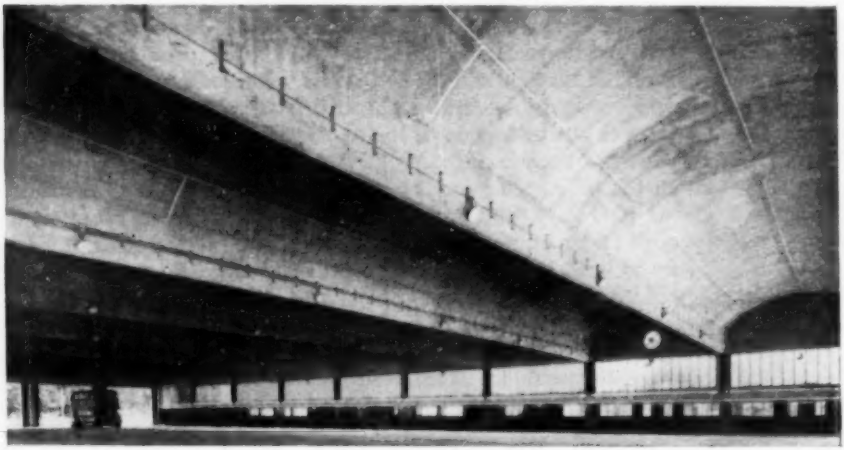
This stress is in excess of that recommended in the British Standard Code of Practice (No. 114—1948) for the Structural Use of Normal Reinforced Concrete in Buildings. It is the opinion of the Council of the Association that standard specifications and codes of practice ought not to be varied to meet temporary emergencies; they are not mandatory and their requirements can generally be modified at the discretion of the engineer responsible for the work. It is not proposed, therefore, to urge the Council for Codes of Practice to amend C.P. No. 114—1948 for this specific purpose, and no departure from B.S. No. 785 is suggested. On the other hand, the London County Council's Building By-laws, which limit the working stress to 18,000 lb. per square inch, are mandatory within the County of London, but their requirements may be modified by waiver under Section 9 of the London Building Act (Amendment) Act, 1935, and a copy of this memorandum has been handed to the London County Council with a recommendation that applications for such a waiver be sympathetically considered.

Further consideration is being given to the question whether, and under what terms, a similar recommendation can be made in relation to mild steel bars of diameters greater than $\frac{1}{2}$ in.

"Shell" Roof of 150-ft. Span.

THE illustration shows the new garage for the Bournemouth Corporation. The main floor is 300 ft. long and 150 ft. wide and is without columns. The clear height is 19 ft. The roof spans 150 ft. and is thought to be the largest shell roof with prestressed concrete edge-beams yet constructed in this country. There are nine vaults each having a radius of 22 ft. 10¾ in., a rise of 6 ft. 9⅝ in., and a chord of 33 ft. The shell is 2½ in. thick except at the ends and springing, where it is slightly thicker. Intermediate edge-beams are 10 in. wide and 5 ft. 6 in. deep. The external edge-beams are slightly larger and have an upstand. The edge-beams were cast in situ and were prestressed by the Magnel-Blaton system with cables of 0·276-in. diameter wire. Each intermediate edge-beam contains 152 wires, the initial tensile stress in which was 125,000 lb. per square inch. An allowance of 15 per cent. was made for reduction of prestress due to shrinkage of the concrete and creep of the wires and concrete. To reduce the shrinkage, each edge-beam was cast in three parts.

The beams and the end-frames were cast before the vaults and were prestressed when the concrete in the vaults had acquired sufficient strength. The vaults were constructed with 1 : 3 mortar applied pneumatically. The reinforcement in the vaults is mild steel bars and steel fabric. The total design load on each bay of the roof is 195 tons, of which 60 tons are live load. Twisted square bars provide continuity between the edge-beams and vaults and, although constructed separately by different methods, they act as a monolithic structure. There are no expansion or contraction joints in the structure, stresses due to change of temperature being allowed for in the design. The columns supporting the roof carry the vertical loads and resist the bending moments due to wind on the whole building.



"Shell" Roof of 150-ft. Span.

The architects are Messrs. Jackson & Greenen, and the consulting engineers Messrs. R. Travers Morgan & Partners. The main contractors are Messrs. James Drewitt & Son, Ltd., and the prestressed work was carried out by Vibrated Concrete Construction, Ltd.

Two Thin-Slab Roofs of 66-ft. Span.

A "SHELL" ROOF AND A PRISMATIC ROOF

THE main roofs of the two structures described have about the same maximum span, that is 66 ft. and 67 ft. respectively, but are different types of reinforced concrete thin-slab construction, one being a "shell" and the other a prismatic structure. The reinforced concrete structures were designed by the British Reinforced Concrete Engineering Co., Ltd., who also supplied the reinforcement. The "shell" roof was designed in association with Mr. C. V. Blumfield.

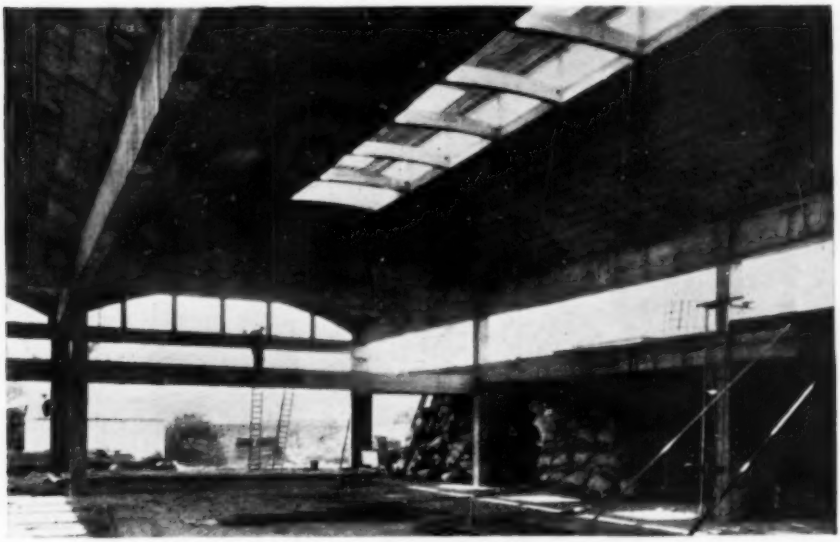


Fig. 1.

A Multiple-span "Shell" Roof.

The new bakery for the Grays Co-operative Society was designed by the Architect's department of the Co-operative Wholesale Society, Ltd., and is being constructed by the Society's Building department. The roof (*Fig.* 1) comprises two longitudinal spans of 66 ft. and one of 28 ft. in each of which there are six transverse circular-arc vaults each 30 ft. wide (*Fig.* 2). At the rear of the main building there is an extension with a roof of 45 ft. span comprising six 30-ft. vaults at a higher level. The reinforced concrete shell is generally 2½ in. thick and is insulated with vermiculite concrete over which there will be waterproofing. The coarse aggregate in the shell is ⅜-in. gravel. The reinforcement (*Fig.* 3) comprises a layer of steel fabric near the top and bottom faces, a central layer diagonally across the vault, and mild steel spacer bars between the layers. Additional bars are provided at the haunches, where the slab is thicker, and around the lantern lights at the crown of each vault. The reinforcement in one vault is shown in the upper part of *Fig.* 8.

The edge beams and valley beams are carried on columns (*Fig.* 1) supported on precast prestressed concrete piles. To prevent bending moments being transmitted from the beams to the columns, hinged joints are provided in some of the external columns at the soffit of the beams. The hinge is formed by bars crossing diagonally at a smaller section

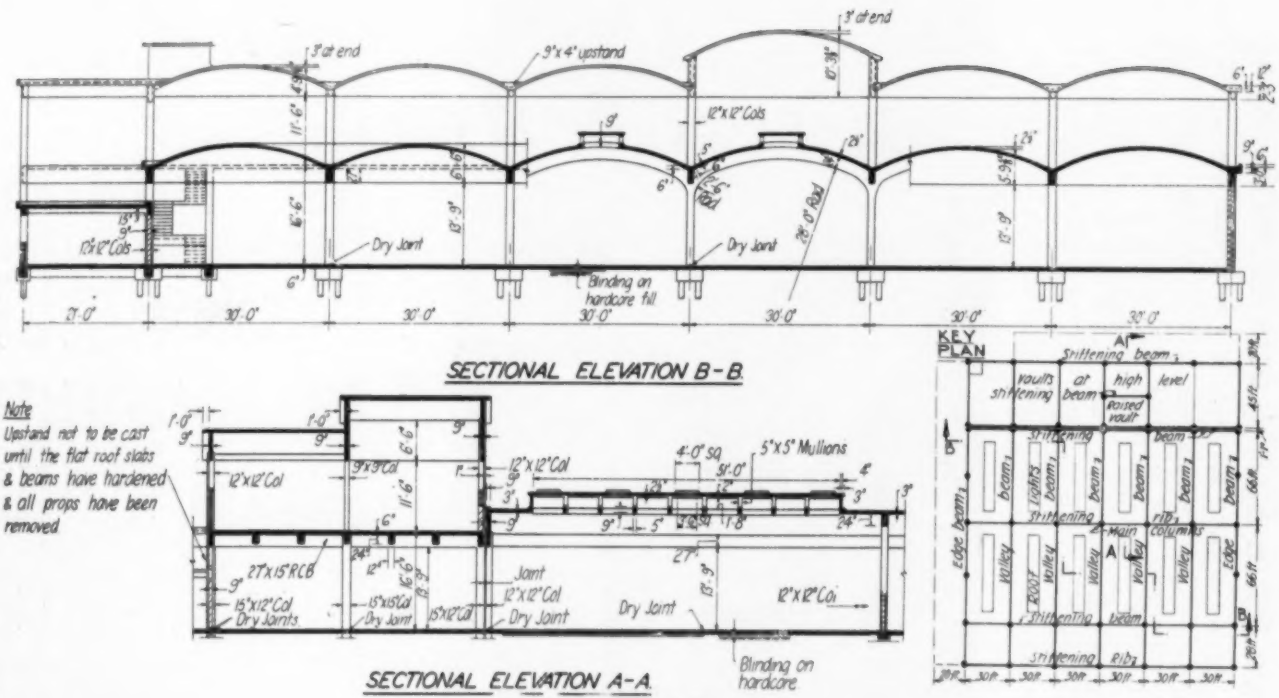


Fig. 2.—Plan and Sections.

of the column, the horizontal gap on each side of the intersection being packed with a flexible joint-filler. Double columns are provided where the main building and the extension meet.

The shuttering for the vaults was of simple construction. Steel tees 1½ in. by 1½ in. bent to the profile of the curved slab support narrow rectangular steel shutter-plates. The tees were held in steel forks (*Fig.* 4) on vertical steel tubes which were supported on scaffolding of steel tubes (*Fig.* 5). The height of the fork is adjustable by turning a nut on a threaded insert in the top of the tube. The shuttering for the edge beams and valley beams is of steel plates (*Fig.* 6) retained by inclined steel tubes.

The vaults were concreted in transverse strips about 6 ft. wide. After the edge and valley beams had been constructed, the next stage was to concrete an area 66 ft. long with a width comprising two complete vaults and about a quarter of the next. The two succeeding stages were to concrete two vaults and a quarter-vault in the next 66-ft. span and the 28-ft. span in line with those concreted in the preceding stage. The remaining three-quarters of a vault, a complete vault, and about one-quarter of the next vault were then concreted. The last stage was to concrete the remaining one and three-quarters of a vault. The wooden template to give the correct thickness of slab is shown in *Fig.* 3. The shuttering to the first vault was not removed until the third vault had been concreted. The shuttering was supplied and erected by Acrow (Engineers), Ltd.

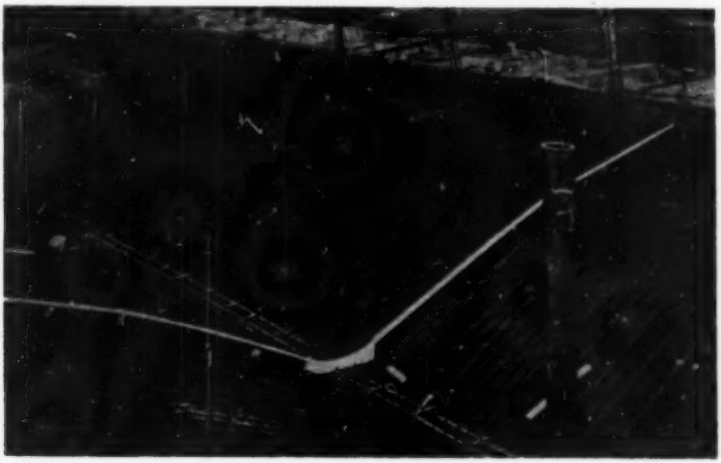


Fig. 3.—Reinforcement of Shell Roof.

A Prismatic Roof.

The reconstruction of the century-old bottling store at Crowley's Brewery, Alton, was carried out to the instructions of Mr. C. F. Morgan, L.R.I.B.A., brewery surveyor of Messrs. Watney, Combe, Reid & Co., Ltd. The contractors were Messrs. F. G. Minter, Ltd. The contractors for the roof covering were Messrs. R. J. Symons, Ltd.

The new roof is a reinforced concrete prismatic structure (*Fig.* 7) the greatest longitudinal span of which is 67 ft. There is also a smaller span of about 32 ft. in line with, but separated from, the longer span. As shown in *Fig.* 9, the roof comprises three full bays and one half-bay transversely, provision being made for a continuation. Each inner bay is about 34 ft. wide and comprises a horizontal valley-gutter slab 6 in. thick, two 4-in. slabs inclined at an angle of 40 deg., and a horizontal top slab varying in thickness from 4 in. to 6 in. Splays are provided at the junctions of the slabs. At the

Fig. 4.—Shuttering for Shell Roof.

outer ends of the roof the support is a vertical fascia wall 6 in. thick, stiffened by a beam 18 in. by 12 in. at the top. At the inner ends of each of the two spans there are diaphragms 7¾ in. thick separated by ½-in. fibreboard. The fascia walls and diaphragms span 34 ft. and extend above the slabs in the valleys. The columns supporting the diaphragms are the only columns inside the building above the ground floor and occur along an interior wall. The columns supporting the fascia walls and the vertical slab at the edge of the outer bay of the roof are in the exterior walls. There is thus a clear working space of about 65 ft. by 120 ft. on the ground floor.

The roof is designed to carry an imposed load of 20 lb. per square foot in addition to its own weight and the insulated roof covering, which comprises a layer of aluminium foil below corrugated asbestos-cement sheeting. At one point provision is made at the junction of the top slab and the adjacent inclined slab for the suspension of a lifting beam designed for a maximum load of 3 tons. The height of 7 ft. 9 in. from the valley gutter to the top of the roof was adopted for reasons other than structural, and

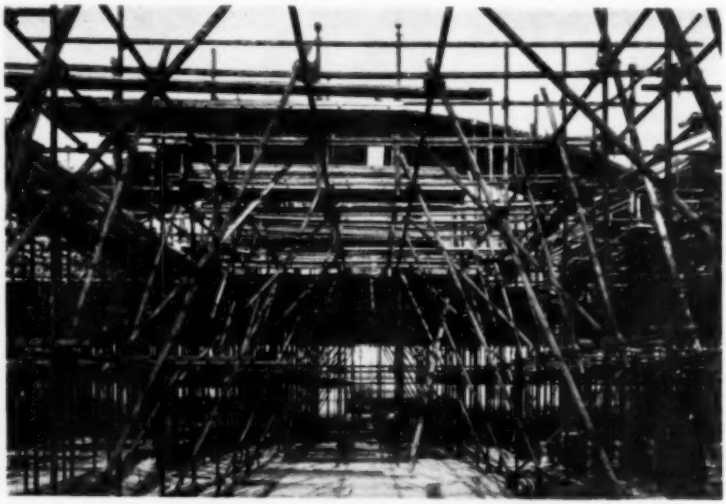


Fig. 5.—Scaffolding for Shell Roof.

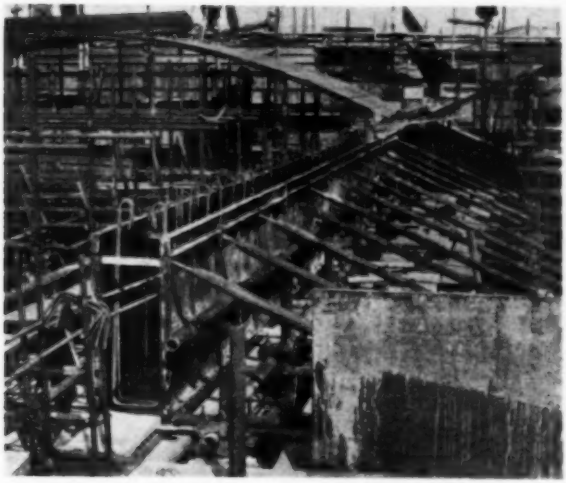


Fig. 6.—Shuttering for Valley Beams of Shell Roof.

results in a roof that could span a greater distance. Consequently the stresses are not great. *Fig.* 10 shows the magnitude and distribution of the longitudinal stresses in the concrete. The reinforcement is shown in *Fig.* 8. Many small bars in the valleys project into the sloping slabs and are bent to conform approximately to the lines of principal stress. Additional bars are provided around the roof lights, which are in the sloping slabs (*Fig.* 11), an advantage of this position being that the glass is cleaned by rain.

The shuttering for the roof slabs (*Fig.* 12) comprised main centering of steel joists shaped to the profile of the roof and supported along the valleys on tubular steel props. Steel ribs spanning between the joists supported steel shutter-plates which had previously been used for the flat-slab ground floor. Timber boards at the valleys and eaves were propped inde-

Fig. 7.—Prismatic Roof.

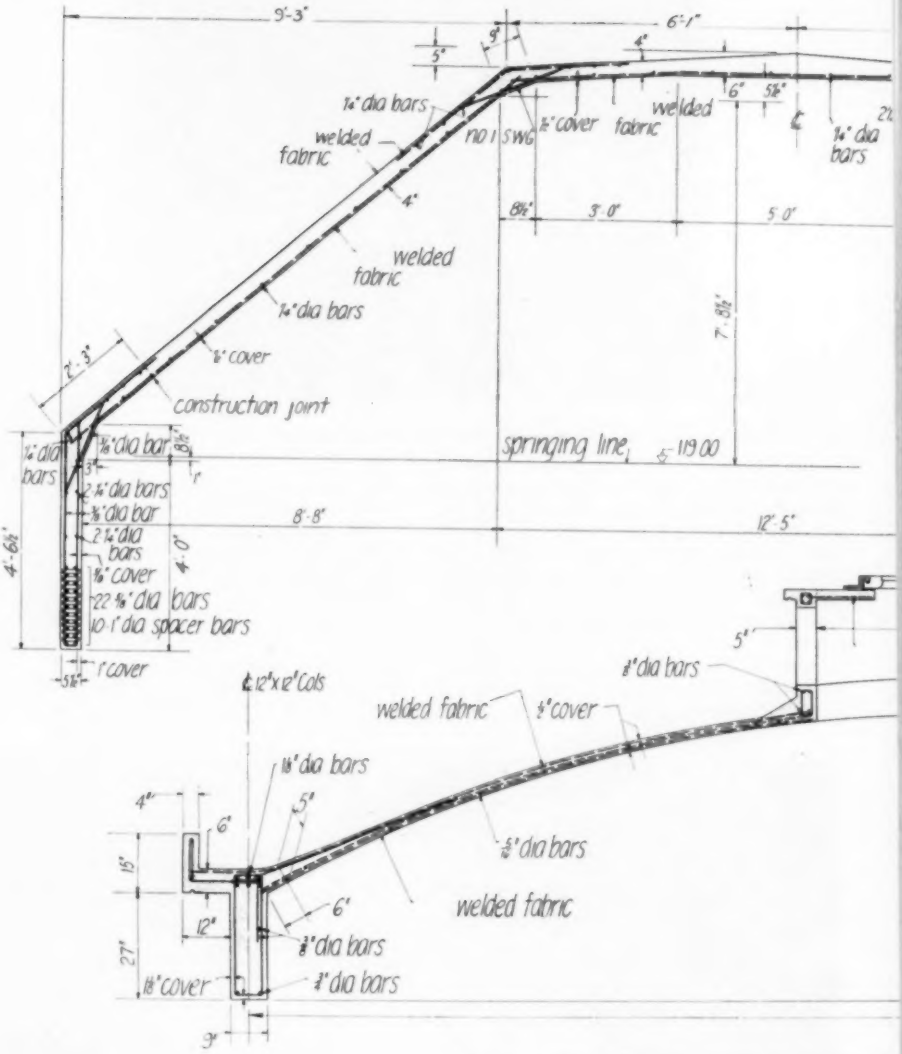
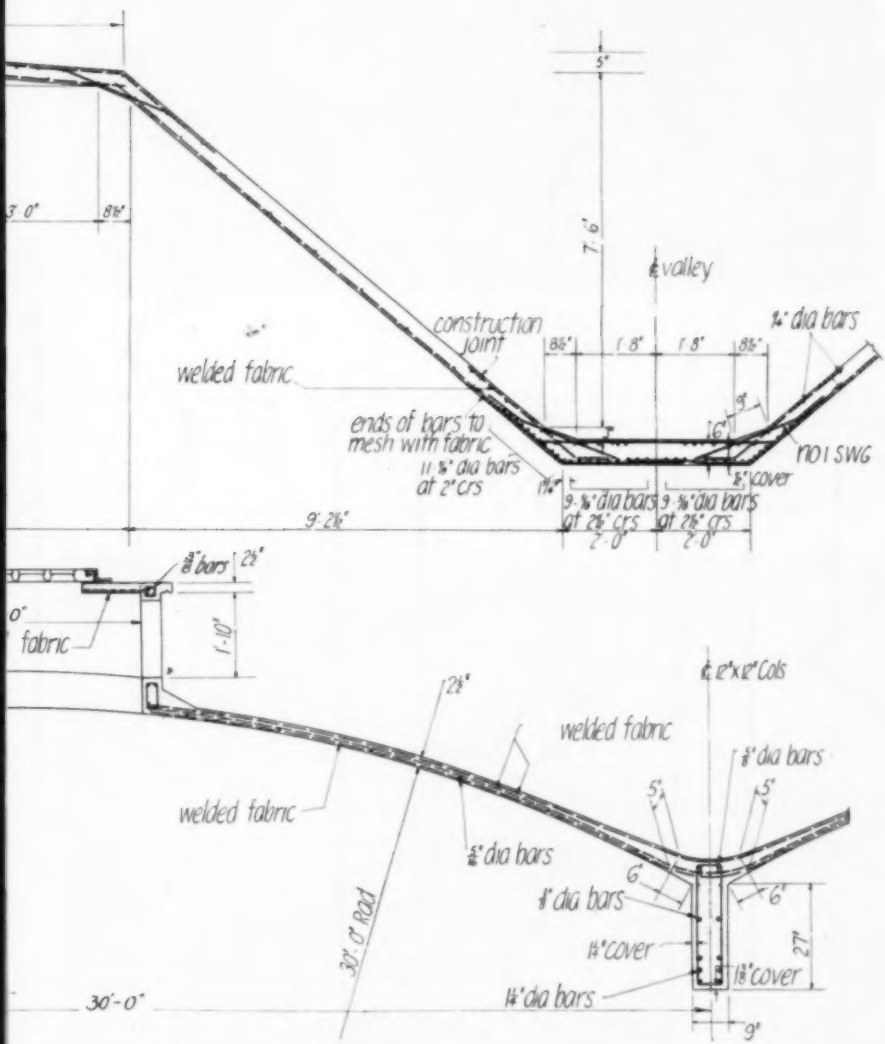


Fig. 8.—Top: Reinforcement in Prismatic



Below : Reinforcement in Shell Roof.

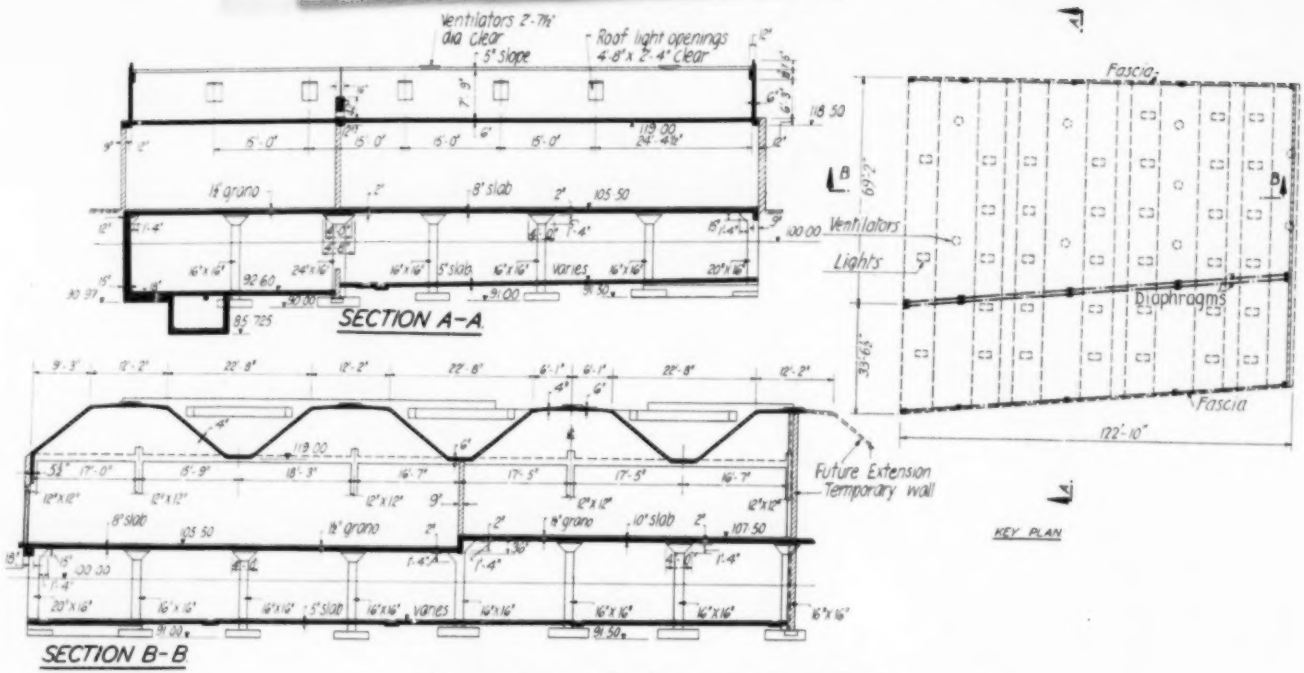


Fig. 9.—Plan and Sections of Prismatic Roof.

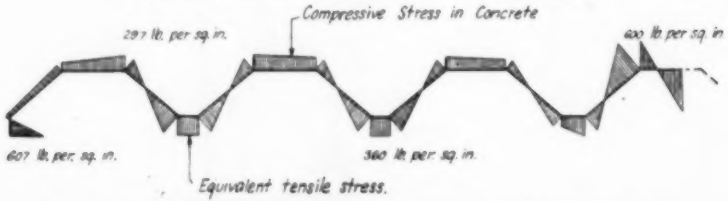


Fig. 10.—Longitudinal Stresses in Concrete of Prismatic Roof.

Fig. 11.—Lights in Prismatic Roof.

pendently of the main centering. The two sets of props along the valleys supported the weight of the roof, and were left in position until all the slabs, diaphragms, and fascia walls had hardened.

The fascias and diaphragms were concreted in two main operations, the first being to place the concrete in horizontal layers up to the level of the soffit of the slabs. The second operation was to concrete the parts of these members projecting above the slabs, in which case vertical

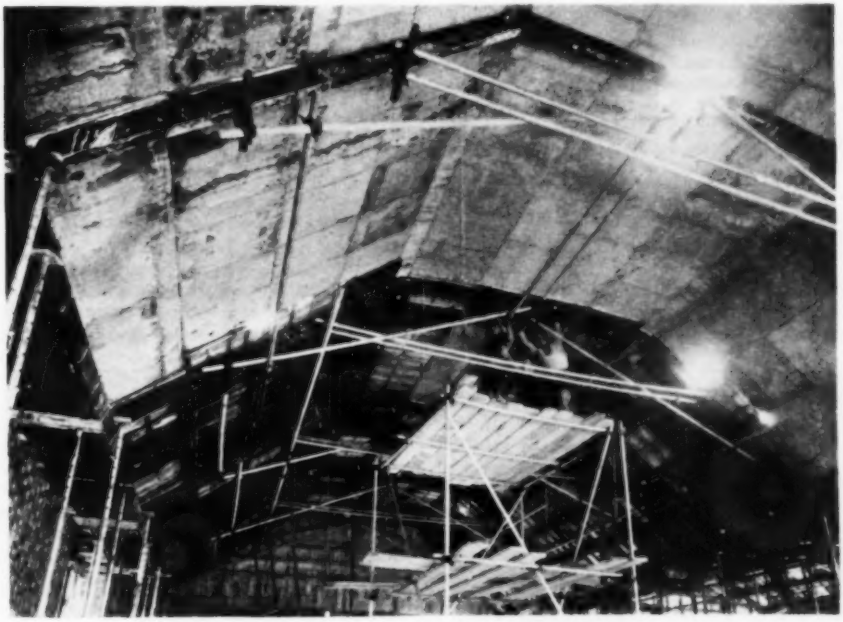


Fig. 12.—Shuttering for Prismatic Roof.

Fig. 13.—Columns for Flat-Slab Floor.

construction joints were made, when necessary, at the centre-line of the valley. The procedure of concreting the slabs gave the least number of construction joints, especially longitudinally. The horizontal slabs in two adjacent valleys and four adjacent sloping slabs for a height of 2 ft. 6 in. above the valleys were concreted first for the entire length between the diaphragm and fascia wall. The remainder of each pair of sloping slabs and the connecting top horizontal slab were then concreted in one operation transversely, but in two or three sections between the diaphragm and fascia wall. The joints between the sections were at right-angles to the span and midway between roof-lights. The volumetric proportions of the concrete are 1 : 2 : 4 and the minimum crushing strength of cubes at 28 days was 3000 lb. per square inch. The shuttering was not removed until cubes had attained a strength of 2000 lb. per square inch, and not until the concrete had hardened in one complete bay, valley to valley, beyond the bay from which the shuttering was to be removed.

There was much other reinforced concrete work in the reconstruction of the building. The floors of the cased-beer store and the bottling store are designed for imposed loads of 2½ cwt. and 3½ cwt. per square foot respectively and are of flat-slab construction supported on 16-in. square columns (*Fig.* 13) at 17 ft. centres. The slabs are 8 in. and 10 in. thick respectively. The floors of the tank room and loading dock are of beam-and-slab construction designed for a superimposed load of 2½ cwt. per square foot. Precast slabs are provided in one bay for access to the tanks.

Analysis of Structures by a Photographic Method.

IN "Photographic Method for Model Analysis of Structures", a paper presented to the Society for Experimental Stress Analysis, New York, Mr. M. M. Rocha and Mr. J. F. Borges, of the Civil Engineering Laboratory, Lisbon, describe a method developed and used at the laboratory. The originality of the method lies in the fact that the displacements of models are measured on a photographic plate on which are photographs of the model before and after deformation. The displacements are produced by applying forces to the model or by applying displacements at certain sections in order to determine influence lines based on the theorems of reciprocity of Maxwell-Betty. Examples of the photographic method are illustrated and, when possible, the results are compared with those obtained by Begg's method and by analytical computations. The photographic method has the advantage that it supplies a record of the tests, and it is claimed to be quicker and easier than other methods. The paper is published in English as Publication No. 18 (1951) of the Laboratorio de Engenharia Civil, Lisbon.

A Cylindrical Prestressed Concrete Tank.

CONCRETE IN WALLS APPLIED PNEUMATICALLY.

A CYLINDRICAL prestressed concrete tank of 250,000 gallons capacity (*Fig.* 1) built for the Crawley Development Corporation by Preload (Great Britain), Ltd., is 49 ft. 6 in. diameter internally and 21 ft. deep, and is the first in this country with stretched wires wrapped mechanically around a wall of pneumatically-applied cement mortar. The roof is a dome having a rise of 6 ft. 3 in.

The method of construction was first to lay a 3-in. slab of plain 1 : 8 concrete on the bottom of the excavation, which is in sandy soil. On this slab was laid a 2-in. floor of pneumatically-applied mortar reinforced with hard drawn steel wire square-mesh fabric weighing 6·1 lb. per square yard. Projecting from the 2-in. slab and extending 3 ft. to 4 ft. vertically into the walls are ½-in. bars at 12 in. centres. The inner part of the wall, which is monolithic with the floor, is 4 in. of mortar applied pneumatically from the inside of the tank against steel shuttering which was erected around the outside to the full height of the wall. The steel shuttering produced a smooth face on which the wire was wrapped. Vertical chases at 2 ft. centres in the inner face of the wall were formed by wooden slats, and each chase contains four wires anchored at the bottom and top of the wall. After the vertical wires were stretched to produce a compression vertically in the wall, the chases were filled with pneumatically-applied mortar. The shuttering for the dome was then erected. The reinforcement in the dome is similar steel fabric weighing about 3½ lb. per square yard, and ½-in. bars project from the wall at 12-in. centres. The dome is 2 in. thick and is also of mortar applied pneumatically. Wire in tension was then wrapped around the outside of the wall and produces a circumferential compression which is greater than the tension which the contents of the tanks tend to produce. The turns of wire are much closer for a distance of 10 in. from the top of the wall than elsewhere, and the effect of this " ring beam " was to impose a thrust at the springing of the dome which was thereby raised off the shuttering and became self-supporting thus enabling the shuttering to be removed. The wire around the tank is embedded in ¾-in. of mortar applied pneumatically. The outside of the tank is finished with cement paint. Pipes to the tank pass under the wall, and connections are made in a sump in the floor.

Pneumatically-applied Mortar.

The mixture of the mortar is 1 part (by volume) of Portland cement to 3½ parts of sand. The grading of the sand is within the following limiting percentages (by weight) : ⅜ in., 100 (maximum) ; ⅛ in., 95 to 100 ; sieve No. 14, 45 to 80 ; No. 52, 5 to 30 ; No. 100, 3 to 8 ; No. 200, 3. Otherwise the sand conforms to B.S. No. 882, but the tensile stress of 1 : 3 mortar specimens was required to conform to B.S. No. 12 (Portland cement). The compressive strength was about 6000 lb. per square inch. The dry mixture of cement and sand was introduced into a stream of compressed air passing through a hose, at the nozzle of which water was mixed with the dry material. The amount of water was such that the surface of the mortar shot at right-angles on to the shuttering had a glossy appearance. The air pressure at the machine end of the hose was not less than 35 lb. per square inch, and the water pressure at the nozzle not less than 50 lb. per square inch. At the joint at the end of each application of mortar, the coat was sloped off at an angle of about 45 deg. and, before recommencing shooting, the sloping face was cleaned and wetted by a blast of air and water. The mortar in the floor was kept wet until the wall was complete.

Prestressing.

The vertical wires were stretched four at a time by an hydraulic jack which was attached to the top and imposed a pull of 2 tons on each wire. The lower end of each pair of wires is continuous around a steel pin embedded in the concrete. The jack, which was designed by the Ministry of Works, comprises four separate pulling devices so that the same pull is exerted on each wire. After stretching the wires, a grip at the top of each wire was tightened so that when the jack was

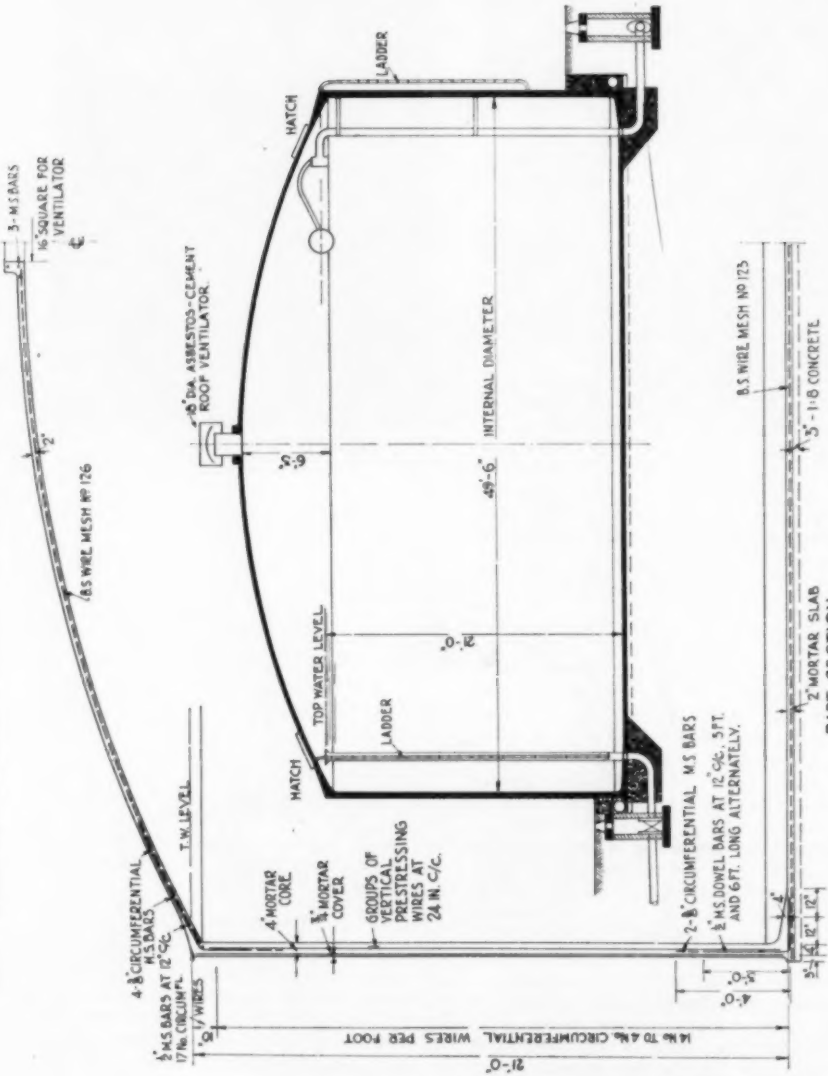

3-M.S BARS
16" SQUARE FOR VENTILATOR
2"
.85 WIRE MESH NO 126
2'DIA ASBESTOS-CEMENT ROOF VENTILATOR
HATCH
LADDER
INTERNAL DIAMETER
49'-6"
6"
0-6"
TOP WATER LEVEL
LADDER
HATCH
T.W. LEVEL
4" MORTAR CORE
3" MORTAR COVER
GROUPS OF VERTICAL PRESTRESSING WIRES AT 24 IN. C/C.
4-8" CIRCUMFERENTIAL M.S BARS
½" M.S BARS AT 12" C/C. 17 No. CIRCUM.
½" WIRES
14 No. TO 4 No. CIRCUMFERENTIAL WIRES PER FOOT
21'-0"
2-8" CIRCUMFERENTIAL M.S BARS
½ M.S DOWEL BARS AT 12" C/C. 5 FT. AND 6 FT. LONG ALTERNATELY.
0.5 WIRE MESH NO 123
2" MORTAR SLAB
3'-1:8 CONCRETE
PART SECTION

Fig. 1.

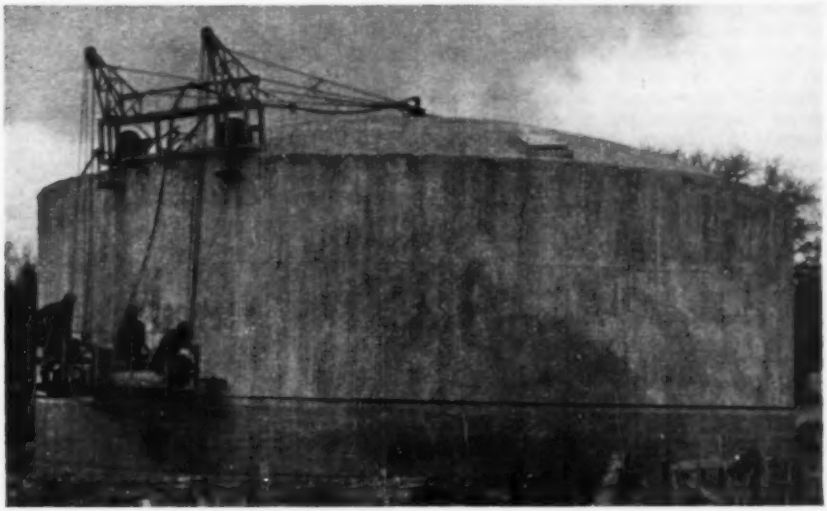


Fig. 2.—Winding Wire Around Tank.

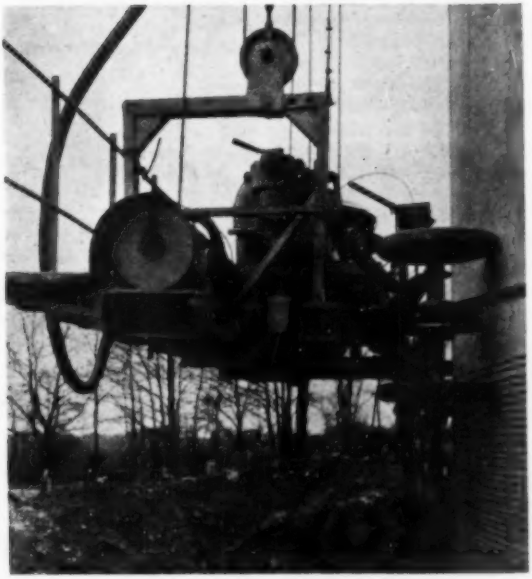


Fig. 3.—Winding Machine.

released the wires remained stretched and imposed a compressive stress of 200 lb. per square inch vertically in the wall.

The circumferential wire was wrapped around the tank by a self-propelled machine (*Figs.* 2 to 4) on a cradle suspended from a trolley. The two vertical rubber-tyred wheels of the trolley travelled on the springing of the dome, and two horizontal guide wheels ran on the wall of the tank. The machine was driven by a motor on the cradle, travel being effected by a toothed wheel engaging a roller-chain wrapped around the tank. The cradle carried the operator and a spool of wire which uncoiled as the machine travelled round. The wire was drawn through a die on the machine, the diameter being thereby reduced. The resistance to the passage through the die provided the tension in the wire (see this journal for May, 1950, page 180). The diameter of the wire was 0·2 in., but passage through the die reduced it to 0·177 in. The pitch of the continuous helix of wire around the tank was controlled by gradually raising the cradle by shortening the suspension wires from the trolley. Adjustment of the level of the die relative to the cradle could also be made, and this method of adjustment was used when wrapping the wires at the bottom of the tank when the die was below the cradle, and at the top when the die was above the cradle.

Each coil of wire was sufficient for about 28 turns. When only a few feet of wire remained to be unwrapped, the machine was stopped, the stretched wire was secured in a grip on an arm (*Fig.* 4) extending from the main vertical post of the machine, and the die was wound forward so that there was no tension in the wire between the grip and the spool. The wire was cut near the grip, the loose end (remaining on the spool) was removed, and the end projecting from the grip was inserted in a double-ended spring chuck. A new coil was placed on the spool, the wire having been previously reduced in diameter to 0·175 in. for a short distance from the end so that it would pass freely through the die, and the end was inserted in the unoccupied chamber of the spring chuck, thus connecting the two lengths of wire together. The die was then wound back to its working position, thereby imposing a tension in the new wire, the grip was released, and the machine set in motion again.

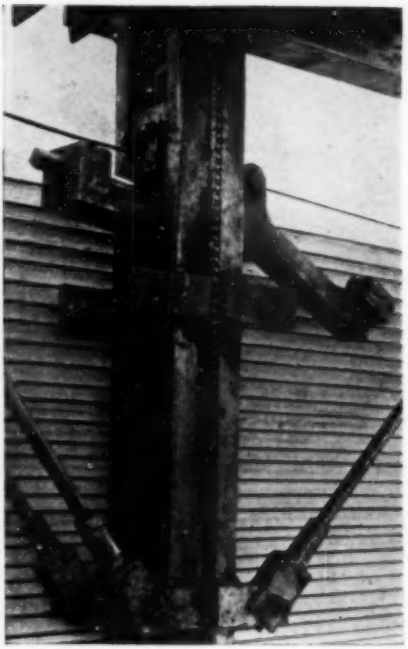


Fig. 4.—Wire-drawing Die and Clamp.

At the commencement of the wrapping process, the end of the wire was held permanently in a wedge-grip embedded in the bottom of the wall. At the end of the process, the wire was passed through a V-slot and a wedge grip was attached to the end. When the wire was cut between the grip and the machine the grip bore against the sides of the slot. Anchorages extending across two or more turns of wire were fixed at intervals so that if a wire broke only a short length became unwrapped.

Passage through the die heats the wire, but the tensile stress in the wire when cool is about 140,000 lb. per square inch. In course of time the stress is reduced by about 28,000 lb. per square inch due to creep. The stress used in the design calculations is therefore 100,000 lb. per square inch which allows a margin for safety. The tensile strength of the wire is

200,000 lb. per square inch and the 1 per cent. proof stress is 180,000 lb. per square inch. The maximum compressive stress in the wall was about 1500 lb. per square inch when the wire was first applied, but, due to the decrease in the stress in the wire, the calculated compressive stress is not less than 720 lb. per square inch when the tank is empty and 50 lb. per square inch when full. At the bottom of the wall there are about fourteen turns of wire per foot, reducing uniformly to four turns per foot immediately below the ring beam in which there are seventeen turns in a depth of 10 in.

This method of construction with a monolithic joint between the floor and wall is said to be suitable for tanks up to 90 ft. diameter, but for larger tanks a sliding joint is provided. Many tanks have been constructed by this method in America.

Book Reviews.

"The Design and Placing of High Quality Concrete." By D. A. Stewart. (London: E. & F. N. Spon, Ltd. 1951. Price 25s.)

IT is to be expected that many books should be written concerning a common synthetic material like concrete, the constituents and properties of which may vary greatly. Often such books are rewritings of much that has been published before, and it is therefore refreshing to read one in which the author has definite, and not always orthodox, views on the subject. The basis of Mr. Stewart's ideas, which are derived from laboratory work and site experience, is consolidation by vibration in which the frequency, period, and other factors are carefully related to the type of concrete and nature of the structural member or product. Existing specifications of concrete are criticised adversely. What is considered by the author to be an improved specification is given which in the first place gives the minimum compressive strength and density required, and is followed by a working specification for the vibration of concrete. Strict adherence to such a specification is not intended for, as the author implies, the engineer should know from experience what to specify to give the result required.

"Nuevo Método de Cálculo del Hormigón Pretensado." By E. Torroja and A. Páez. (Madrid: Instituto Técnico de la Construcción y del Cemento. 1951. Price 50 pesetas.)

THE first part of this book, which occupies about two-thirds of the total of about 300 pages, describes a new method of designing prestressed concrete. The method is based on the ultimate resistance, and the special condition is that the compressive and tensile stresses in the concrete and the tensile stress in the steel attain simultaneously values equal to the compressive and tensile strengths of the materials, so that, even just before failure, the whole of the concrete would contribute to the strength of the member. The theory disregards Hooke's law. Calculations are based on stress-strain diagrams which are close approximations to the results of tests. The ability of steel to be strained beyond its elastic limit enables it to be used beyond this limit and, although such a condition implies a low "elastic" factor of safety, the authors state that the structure is safe. They claim that a theoretical approach has been found which permits exploitation of the available reserve strength. In view of the many uncertain factors included in such calculations it is doubtful whether the method will commend itself to all engineers. The second part contains descriptions of prestressed concrete structures, most of which have been described elsewhere.

"La Pathologie du Béton Armé." By H. Lossier. (Paris: Dunod. 1952. Price 480 fr.)

FAULTS in the design and construction of some reinforced concrete structures and foundations resulting in defective structures are dealt with in detail and illustrated by sketches and photographs. The probable causes are examined, remedial measures described, and means of avoiding similar occurrences considered. Although the book contains only 100 small pages, the subject-matter is extremely valuable because structural defects are rarely publicised in this manner.

An Early Reinforced Concrete Bridge.

OUR illustrations are of a reinforced concrete foot-bridge designed by François Hennebique and built at Liege in the year 1904. The total length of the bridge is 263 ft. and the central span is 180 ft. long.

Referring to this bridge in his inaugural address at the Congress on Prestressed Concrete held in Ghent in September last, Professor Gustave Magnel said that in the early days of reinforced concrete all design was empirical, and the design of this bridge was the result of intuition rather than of calculation. The bridge was so light that it was doubtful if it would be built to-day; the stresses due to the loads, shrinkage, changes of temperature, and other causes were much higher than those adopted to-day in the elastic theory. "However," continued the speaker, "this elegant bridge has been in use for nearly fifty years and is still in fine condition. The bridge knows the laws of nature better than the most clever professor, and it knows perfectly well how to behave under all conditions. So let us pay tribute to those who were bold enough to use reinforced concrete when scientific knowledge was poor, and when it was not possible to design structures with a given factor of safety at a minimum cost. All technical progress is conditioned by the courage of a few. Who were right—the Americans who built flat-slab floors without a theory and experimented until they developed a satisfactory design, or the Europeans who waited patiently for formulæ? These formulæ were wrong because not all the assumptions were valid—but engineers often prefer an incorrect formula to a perfect experiment."

A Jetty for Large Tankers on the Thames.

PRESTRESSED CONCRETE BEAMS AND SUSPENDED FENDERS.

A JETTY (*Fig.* 1), which is being built at Thames Haven on the Thames estuary by London & Thames Haven Oil Wharves Ltd., is designed for 28,000-tons oil tankers which bring crude oil for the nearby refinery at Shell Haven. The structure, which is L-shape in plan, comprises an approach (*Fig.* 2) 414 ft. long and 23 ft. wide and a jetty-head (*Fig.* 3) 240 ft. long and 40 ft. wide. The construction is unusual in that prestressed concrete beams are used in the deck, and the fenders, also of prestressed concrete, are of a horizontally-suspended type.

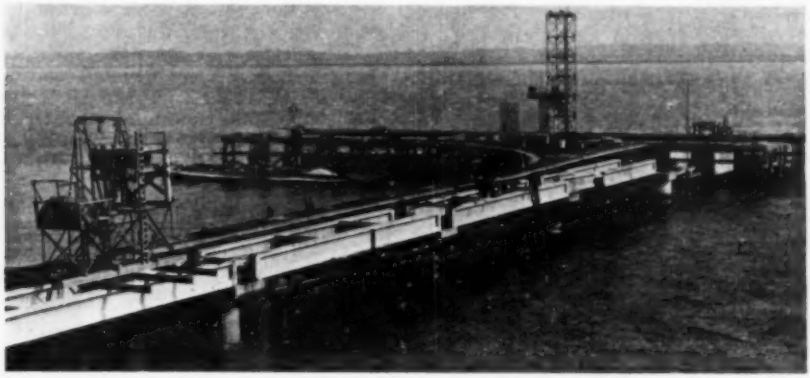


Fig. 1.—Jetty in Course of Construction.

The principal temporary works are a casting yard on the river bank and a temporary staging extending from the shore along the downstream side of the approach and along the front and back of the jetty-head. A crane-track from the casting yard extends the entire length of the staging and carries four 7-tons steam locomotive cranes. At the casting yard the crane-track is on a concrete slab because of the poor nature of the ground, and on timber trestles on concrete foundations between the casting yard and the river bank. The staging in the river comprises 182 12-in. by 12-in. broad-flange beams 70 ft. long driven as piles with timber bracing and steel tie-rods; the crane-track is carried on 24-in. by 7½-in. steel beams.

Substructure.

The substructure of the approach and jetty-head consists of hollow reinforced concrete cylinders filled with concrete. The cylinders, which were precast, are in sections each 8 ft. long and have ogee joints. The external diameter is 8 ft. 3 in. and the thickness 4½ in. The bottom edge of the lowest section has a steel cutting-edge. The sections were brought to the site by road, stored in barges until required, and were erected in a steel guide-frame (*Fig.* 4) to maintain the alignment while being sunk. The height of the cylinders varies with the level of the ballast on which they are founded; the greatest height is 80 ft., although while the cylinders were being sunk the height was 92 ft.

A temporary section with a thicker wall and a steel cap was placed on the top of each cylinder to carry the platform for the cast-iron kentledge, the action of which in sinking the cylinders was assisted by excavating inside the cylinders with a ½ cu. yd. orange-peel grab, the spoil being dumped into barges. The average weight of kentledge required to sink the cylinders 24 ft. to 36 ft. was about 54 tons. After each cylinder was sunk, mild steel reinforcement was placed in it and it was filled with concrete to above low-water level. Concreting proceeded in a con-

tinuous operation through a tremie suspended from a steel tower (seen on the left-hand side of *Fig.* 1) which was moved by two steam cranes from one pier to the next.

The Approach.

The approach (*Fig.* 2) is supported intermediately on three piers of one cylinder each, a pair of cylinders at the jetty-head end, and two 18-in. diameter bored piles in the river bank. Cast-in-situ reinforced concrete caps 15 ft. by 9 ft. and 30 in. deep are provided on the single cylinders to carry the deck beams.

In each of the four spans of the approach, which have a clear length of 77 ft., there are two longitudinal prestressed concrete I-beams. The beams, which are 4 ft. 6 in. deep, are at 12 ft. centres transversely and are connected by cast-in-situ reinforced concrete transverse webs 9 in. wide at each of the third-points and 15 in. wide over the piers. The deck comprises 5-in. precast slabs 2 ft. wide and 23 ft. long, which cantilever about 5 ft. beyond the edge of the main beams to which they are secured by fish-tail bolts.

Each longitudinal beam was made in three sections about 25 ft. long, which were placed in position in the jetty on a temporary staging. A gap 9 in. wide was left between the ends of each section and was filled when the cast-in-situ transverse webs were cast. The three sections were formed into one structural member by the prestressing wires which were threaded through 1 $\frac{1}{16}$-in. diameter ducts formed by inflated rubber tubes when the sections were cast. Each length of tube formed two ducts, since it was brought through the end shutter and passed around a wooden template as shown in *Fig.* 5. The tubes were held in position by rectangular concrete blocks threaded on them and secured by tying-wire to the vertical stirrups in the beam. Before the concrete was placed the tubes were inflated, and when the concrete had hardened they were deflated and withdrawn for re-use. A precast concrete block (*Fig.* 6) containing the cones in which the prestressing wires are anchored is embedded in the end of each beam. A gap 6 ft. long was left between the ends of the main beams to enable the jacks to be manipulated. After pre-

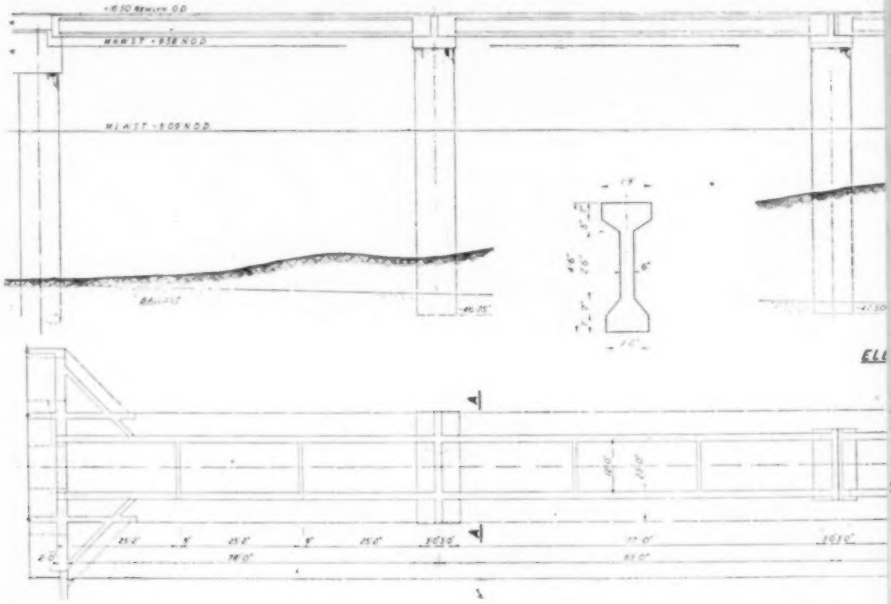


Fig. 2.—D

stressing the 6-ft. gap was filled and the transverse 15-in. web was cast in situ. Four ⅞-in. bars projecting from the ends of the precast beams tie the beams to the cast-in-situ construction over the piers.

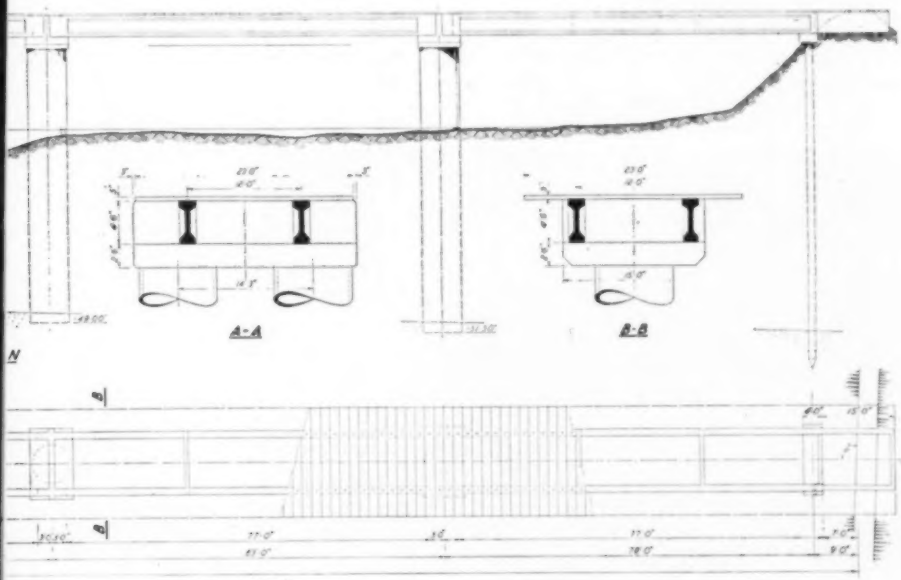
The beams were cast in sections in timber moulds (*Fig.* 7) erected on a timber grid laid on the ground in the casting yard. The concrete was mixed in a ½ cu. yd. non-tilting drum mixer discharging into a ½ cu. yd. bottom-opening skip which was lifted by a crane and emptied on to a stage from which the concrete was placed in the moulds by shovels. The concrete was consolidated by vibration, and rapid-hardening Portland cement was used. The deck slabs were cast in wooden moulds, from which they were lifted after about four days. The slabs, which weigh up to 1¼ tons each, were picked up by the crane at the casting yard and carried to the shore end of the temporary stage, where another crane transported them to the work.

An expansion joint (*Fig.* 8) is provided over one of the piers of the approach. Two transverse webs are separated by a gap 1 in. wide. One pair of beams and one web are cast monolithic with the head of the pier, the other pair of beams and the other web being freely supported on the pier. A roller joint is provided under each beam by an upper and lower 1½-in. manganese steel plate bolted to the concrete of the beam and pier respectively, with a 5-in. diameter manganese steel roller 21 in. long between.

The Jetty-head.

The support of the jetty-head (*Fig.* 3) is provided by three dolphins each comprising six cylinders, three at the front and three at the back. Each pair of the six cylinders is connected by a transverse prestressed concrete box-beam 40 ft. long, 9 ft. wide, and 8 ft. 6 in. deep. A pair of fenders is provided at each dolphin and lies between the box-beams. The dolphins are connected by five parallel prestressed concrete I-beams each 52 ft. long on which are laid 4-in. precast slabs 9 ft. long and 2 ft. wide.

Each box-beam (*Fig.* 9) comprises six precast open rectangular reinforced concrete sections, each 2 ft. 8 in. wide with 7-in. walls, laid between two cast-in-situ



of Approach.

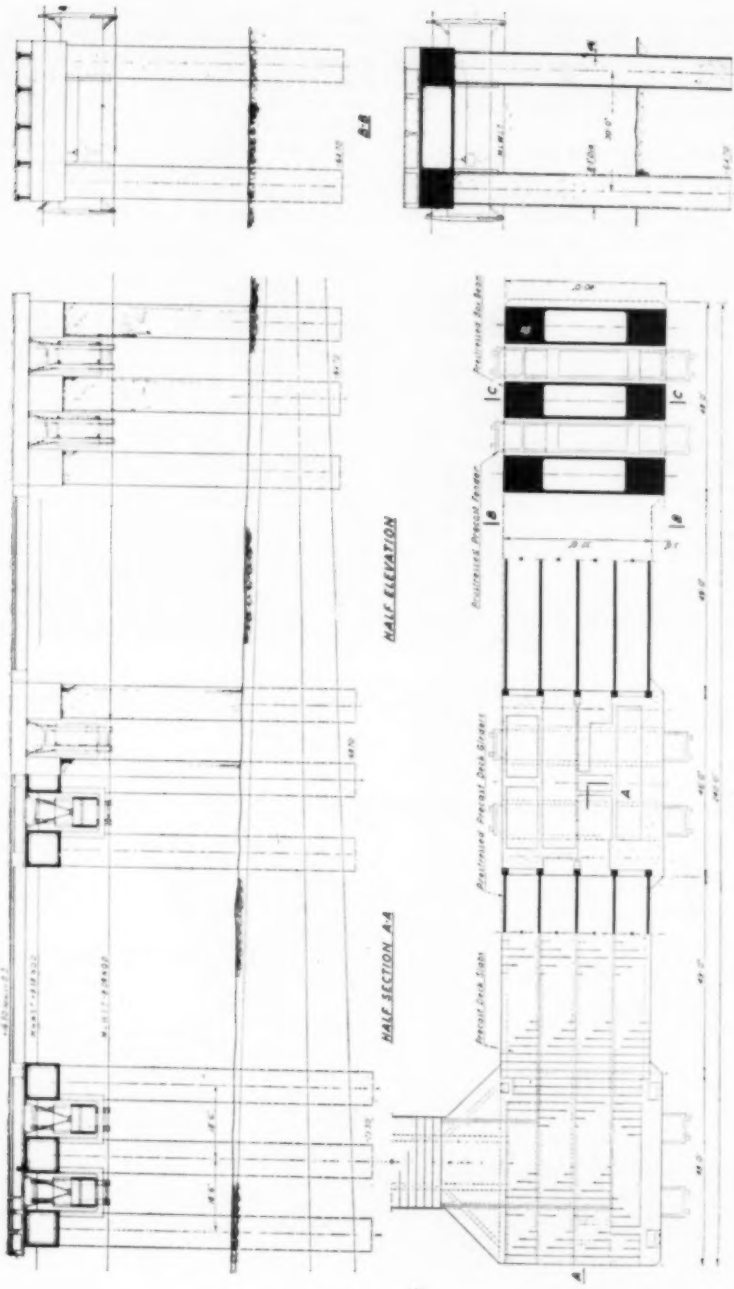


Fig. 3.—Plan, Elevation, and Sections of Head of Jetty.

heads on the cylinders. The sections were erected with a 6-in. space between each, the space being filled in situ. Ducts in the walls of the sections enabled cables to be passed through from one head to the other. Stretching and anchoring of the wires formed the sections into a hollow prestressed concrete beam spanning between the cylinder heads, and with the cylinders form a rectangular frame. Horizontal forces acting on the jetty-head are thereby resisted at each dolphin by three parallel transverse frames.

The 52-ft. beams between the dolphins were cast complete and prestressed in the casting yard. Three mould-bottoms and one pair of mould-sides were provided. A platform over the space for the three moulds enabled the concrete to be placed from above from a skip. The beams were prestressed ten days after casting, when the crushing strength of the concrete was not less than 6000 lb. per square inch. When each beam had been prestressed, it was lifted by two cranes and carried to the jetty-head.

Concrete for filling the cylinders and for cast-in-situ work in the jetty-head was mixed in a barge (*Fig.* 12). Fine and coarse aggregates were stored at opposite ends of the barge, and cement was stored under cover along one side. A ¼ cu. yd. non-tilting drum mixer was placed amidships and discharged into 1½ cu. yd. bottom-opening skips which were taken by crane from the barge to the point of placing.

Fig. 4.—Temporary Trestle and Cylinder for Approach.

The Fenders.

There are six suspended fenders each of which is a prestressed reinforced concrete box 48 ft. long, 7 ft. 6 in. wide, 14 ft. deep at the ends and 6 ft. at the

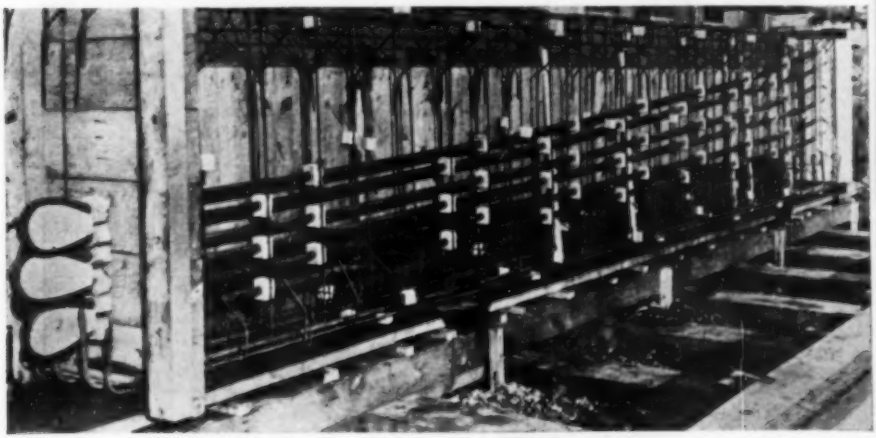


Fig. 5.—Inflated Rubber Tubes forming Cores for Prestressing Cables.

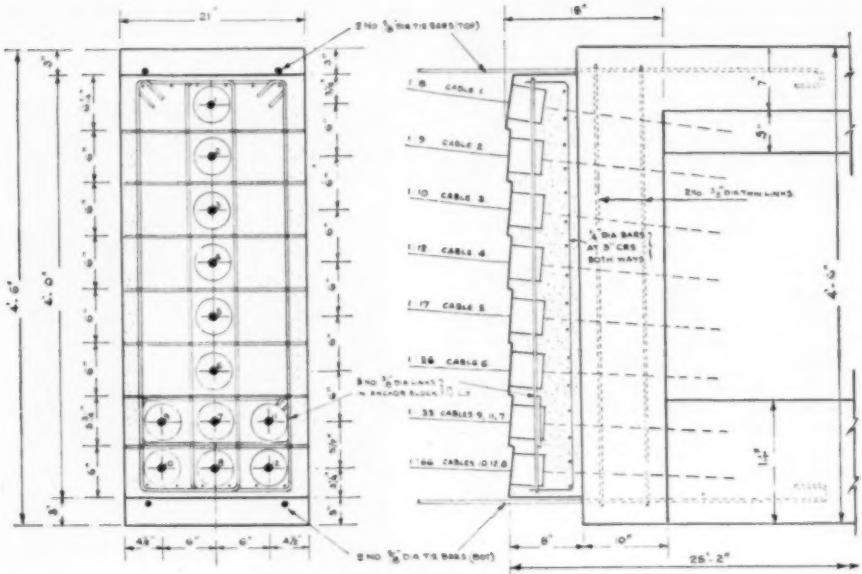


Fig. 6.—Precast Anchorage Block.

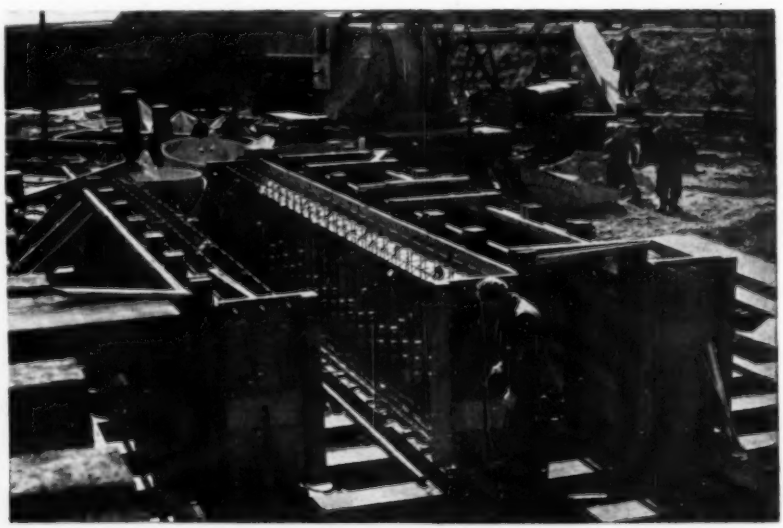


Fig. 7.—Casting Prestressed Concrete Beams.

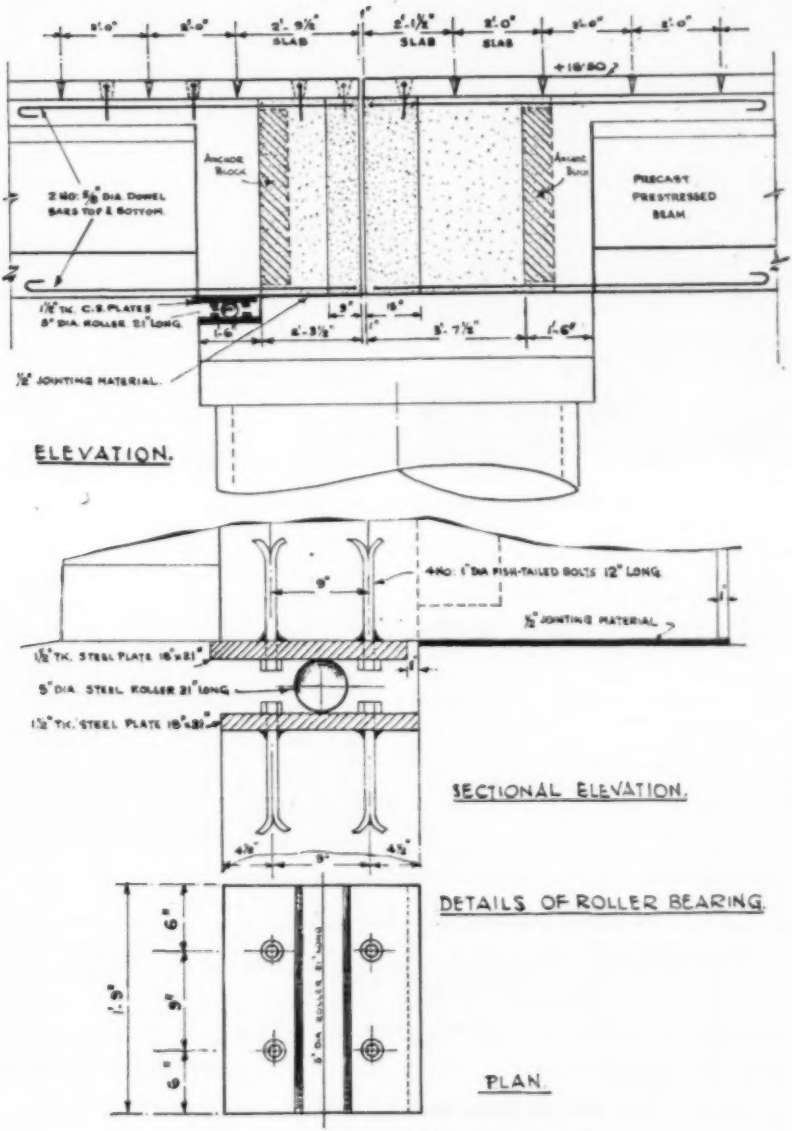


Fig. 8.—Expansion Joint in Approach.

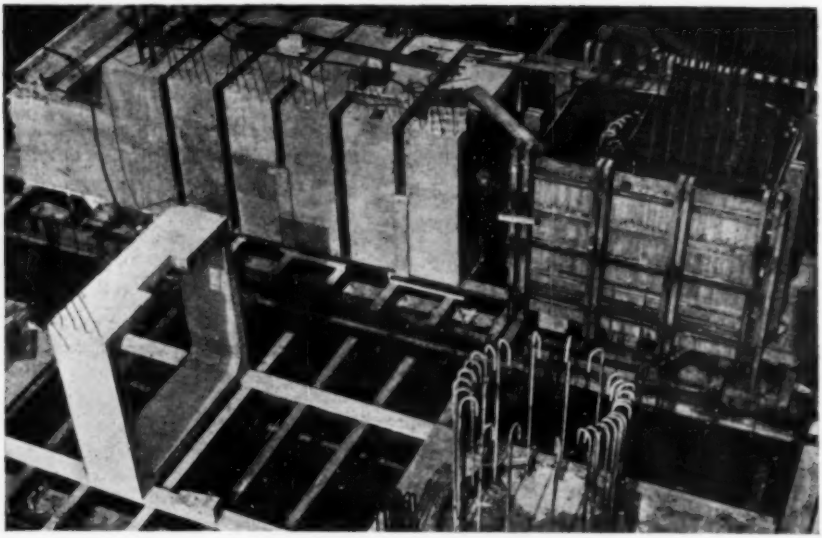


Fig. 9.—Prestressed Concrete Box-Beams in Head of Jetty.

middle, and weighs 56 tons when dry and 30 tons when immersed in water. Each fender is suspended below the deck of the jetty-head by four 2¼-in. diameter chains in such a position that it projects 6 ft. in front and 2 ft. at the back of the jetty-head (*Fig.* 10). When struck by a vessel the fender moves inwards and upwards; the energy each fender can absorb, when immersed, is 90 ft.-tons.

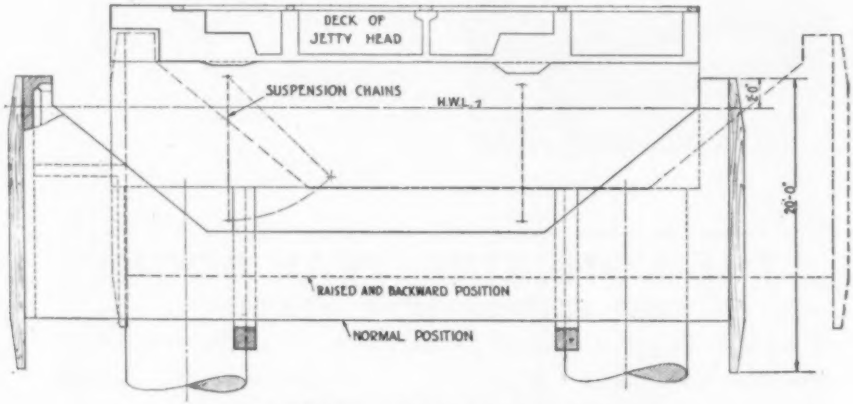


Fig. 10.—Diagram showing Action of Fenders.

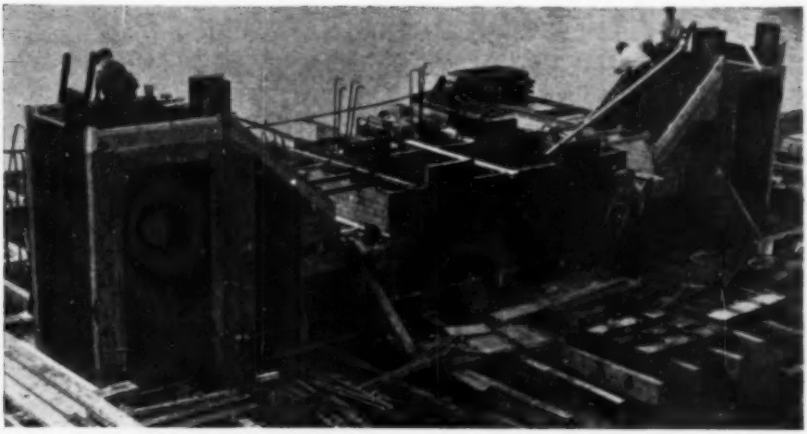


Fig. 11.—Suspended Fender in Course of Construction.

Fig. 12.—Mixer on Barge.

The fenders were cast on the dolphins in steel shutters (*Fig.* 11). After prestressing they are lowered by chain-blocks and are then suspended below the deck. Two timber rubbing-pieces are provided at each end.

Construction of the jetty commenced in May, 1950, and is expected to be completed in 1952. The cost will be about £300,000. The joint consulting engineers are Mr. Thos. C. Rolland and Messrs. L. G. Mouchel & Partners, Ltd., who, in conjunction with Port and Harbour Fenders, Ltd., designed the fenders which are based on the patented design of Professor A. L. L. Baker. The contractors are Messrs. John Mowlem & Co., Ltd. The precast cylinders were made by Spun Concrete, Ltd. The beams were prestressed by the Freyssinet system.

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A Prestressed Concrete Sewage Pit.

A PRESTRESSED concrete pump pit for sewage has been built at the East Greenwich works of the South Eastern Gas Board. The pit (*Fig.* 1) is 16 ft. deep and about 22 ft. by 23 ft. It comprises two chambers, one of which contains the pumps and is permanently dry. The other chamber acts as a reservoir for sewage and at times may be full. The structure is therefore subjected to internal liquid pressure in one chamber, and external pressure from ground water on all sides. The provision of a prestressed concrete structure resulted in a reduction in the amount of steel and shuttering, as it was possible to precast the walls and roof.

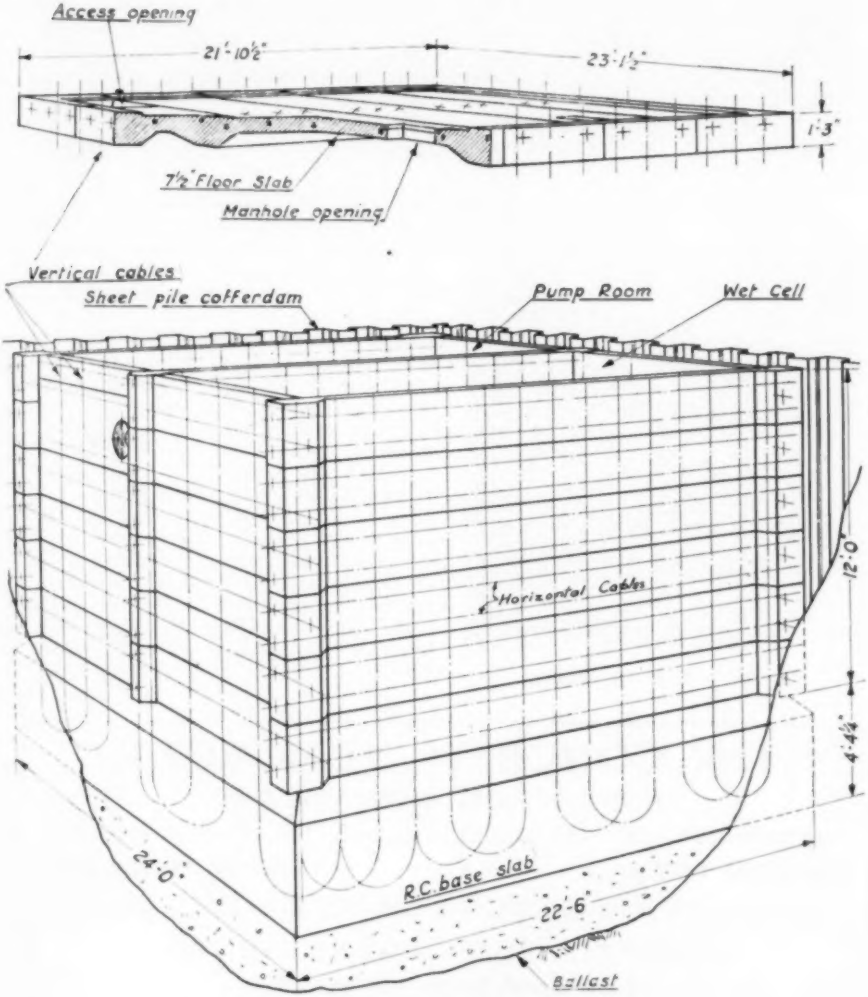

Fig. 1.—Prestressed Concrete Sewage Pit.

The reinforced concrete foundation slab was cast in situ and in it are embedded the looped ends of the vertical prestressing cables. The walls are constructed of slabs in six horizontal courses each about 2 ft. deep. Each course comprises three transverse slabs forming the end walls and division wall, and four longitudinal slabs forming the two longitudinal walls. The slabs, which are 7½ in. thick, were cast flat and those for the end walls and division walls were prestressed before erection. The slabs for one course were placed in position over the structure and cables were threaded through the ends of the transverse slabs and through the length of the longitudinal slabs. Stretching these cables prestressed the longitudinal slabs. The whole course of slabs was then lowered down the vertical cables on to the foundation or on to the course previously laid, and successive courses were constructed and lowered until the walls were complete.

The floor (or cover) of the pit comprises 7½-in. precast concrete slabs each about 3 ft. wide prestressed by longitudinal cables before being placed in position. After erection of the slabs, transverse cables were threaded through holes formed in the slab and stretched, thus making the slabs an integral structural member 21 ft. 10½ in. by 23 ft. 1½ in. The vertical cables from the walls pass through holes in the outer edge of the cover and were stretched and secured to anchorages in the cover, thus fixing the cover to the walls and prestressing the walls vertically.

The pit was prestressed by the Freyssinet method. Each cable contains twelve 0·2-in. wires having a tensile strength of 100 tons to 110 tons per square inch. The stress in the wire is about 60 tons per square inch. The slabs are thickened at the ends to accommodate the anchor cones and some secondary mild steel reinforcement. The holes through which the cables pass were formed by an inflated rubber tube placed in the mould and withdrawn after the concrete had hardened.

The joints between the slabs were left open for a width of about ¾ in. and were filled by ramming in dry mortar. This method produces a joint of high strength but one that is not entirely watertight as damp patches appeared at some intersections of horizontal and vertical joints. The damp patches were treated with a waterproofing compound.

The pit and cover were designed as a monolithic box, the cables being placed eccentrically to counteract the larger horizontal bending moment. The effect of indeterminacy on the value of the prestress was in part avoided by prestressing the more important of the members, that is the transverse walls, when in a statically-determinate condition. The indeterminant effects of prestressing of the longitudinal walls was almost eliminated by placing the cables according to the concordant-cable theory of M. Guyon. The vertical cables, being central in the slabs, produced no indeterminate effects.

The openings in the walls and cover were formed at the time of casting the slabs, and some peripheral reinforcement is provided. The cables are bent around the openings, it being assumed that, since the forces due to the loads are obliged to deviate at the opening, the prestressing forces acting counter to the load forces would do likewise.

The pit was designed by the Prestressed Concrete Co., Ltd., and constructed by the Demolition & Construction Co., Ltd.

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Prestressed Concrete Runway at London Airport.

THE prestressed concrete taxi-track about 355 ft. long and 120 ft. wide at London Airport is now complete. The design (*Fig.* 2) was described in this journal for June, 1949, and is similar to that of a runway at Orly, France. The track is designed for a load of 72 tons on two wheels at 55-in. centres and having a tyre-pressure of 120 lb. per square inch. (The runway at Orly was satisfactory under a test load of 130 tons applied on an area of 3 ft. 3 in. diameter and under a load of 90 tons repeated fifty times.)

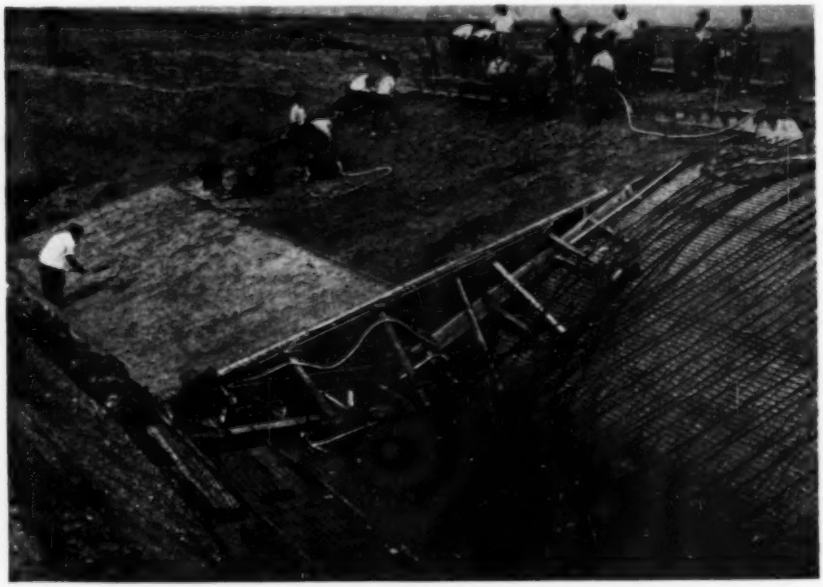

Fig. 1.—Abutment during Construction.

The track is composed of precast unreinforced concrete slabs most of which are 2 ft. 11 in. square and 6½ in. thick, and is divided into a number of 45-deg. triangles separated by gaps in which small steel rollers are placed vertically. At each end of the track there are prestressed concrete abutments. Transverse cables (Type 1 in *Fig.* 2) are placed in grooves at the joints between the precast slabs. The transverse joints on one side of the diagonal roller-joints were in the first instance offset by 1 in. from the corresponding joints on the other side in order to tend to cause movement of the triangles when the transverse cables are stretched. Overall longitudinal movement, that is elongation of the track, is prevented by the abutments, so that the effect of stretching the cables tends to produce movement in the direction of the width of the track. This transverse movement is opposed by the reactions of the adjoining slabs. Thus a balancing stress is developed in the direction of the longitudinal axis of the track. The transverse stress is about 550 lb. per square inch, but the longitudinal stress transmitted to the abutments is less as it is reduced by friction of the slabs on the ground.

Each transverse cable comprises thirty 0·2-in. hard drawn steel wires wrapped in bitumen-coated paper. The tensile strength of the wires is 90 tons to 100 tons

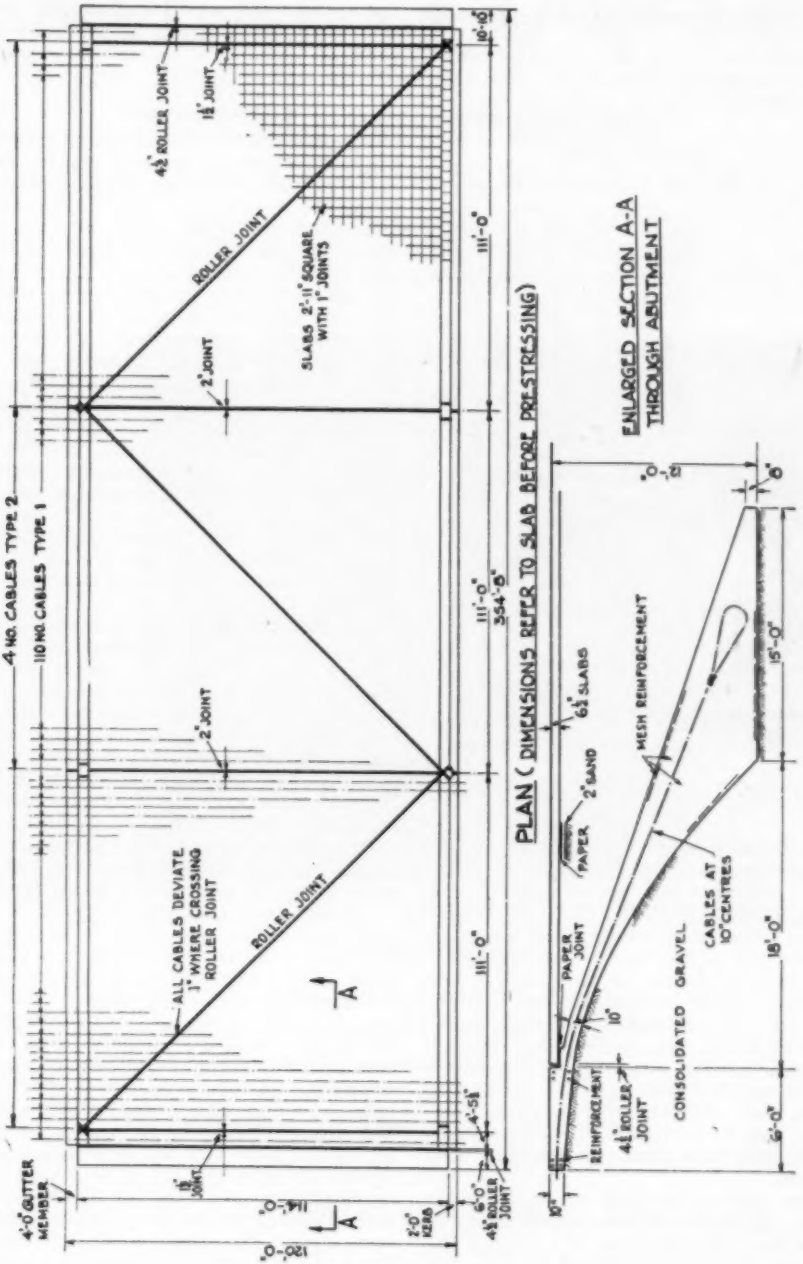
4 NO. CABLES TYPE 2
110 NO. CABLES TYPE 1
4½" ROLLER JOINT
1½" JOINT
ROLLER JOINT
SLABS 2'-11" SQUARE WITH 1" JOINTS
2" JOINT
ALL CABLES DEVIATE 1" WHERE CROSSING ROLLER JOINT
111'-0"
354'-6"
4'-0" GUTTER MEMBER
1" JOINT
2'-0" KERB
6'-0"
120'-0"
PLAN (DIMENSIONS REFER TO SLAB BEFORE PRESTRESSING)
ENLARGED SECTION A-A THROUGH ABUTMENT
6½" SLABS
MESH REINFORCEMENT
2" SAND
PAPER
PAPER JOINT
REINFORCEMENT
4½" ROLLER JOINT
CONSOLIDATED GRAVEL
CABLES AT 10" CENTRES
15'-0"
18'-0"
6'-0"

Fig. 2.

per square inch. To prestress the track, the stress in the cables was 70 tons per square inch. Each cable in the two abutment blocks comprises twelve 0·2-in. wires and the cables are at 10-in. centres.

Precast Slabs.

The proportions (by weight) of the concrete in the precast slabs, of which there are about 4600, are 1 part of rapid-hardening Portland cement, 1·64 parts of fine aggregate, 0·94 parts of coarse aggregate from $\frac{1}{16}$ in. to $\frac{3}{8}$ in., and 2·11 parts of coarse aggregate from $\frac{3}{8}$ in. to $\frac{3}{4}$ in. The concrete was consolidated by a method that the manufacturers believe to be new, that is by applying vibration to the top surface of the concrete while the mould was on a vibrating table. The water-cement ratio was 0·4, and no difficulty was experienced in obtaining a satisfactory finish. The compressive strength of 6-in. cubes at seven days was specified to be not less than 3500 lb. per square inch, and the strengths obtained were from 5000 lb. to 7000 lb. per square inch. The slabs were cured by spraying with water.

Fig. 3.—Placing the Slabs.

The slabs were cast in steel moulds with the top face downwards, and, to turn the slabs over without damaging the arrises, each slab was lifted by a pair of tongs at the end of the arms of which there was a rotatable cone, so that the slab, when raised clear of the casting bed, could be turned over and deposited with the top uppermost.

Construction.

The excavations for the abutments were trimmed accurately to conform to the curve required. The prestressing cables in the abutments have loops of bare wire at one end (*Fig.* 1) to provide an anchorage in the mass concrete forming the lower part of the abutment so that the cables could be stretched without slipping after the concrete had hardened. At the other end the cables are anchored in cones in a transverse reinforced concrete beam. Each of the two abutments contains 360 cu. yd. of concrete which was placed in three sections. *Fig.* 1 shows the first part of one abutment almost completed and some of the cables in position for the second part. The filling over the abutments is gravel, the lower part being placed in 4-in. layers until the surface was wide enough to permit the use of a roller with rubber tyres, when 9-in. layers were placed.

The foundation for the precast slabs was prepared by two different methods because of the experimental nature of the track. For one half, the top soil was removed to expose the underlying brick-earth upon which was deposited not less than 2 in. of sand and a layer of bituminous paper on which the slabs are laid. For the other half, the excavation was taken down an additional 8 in., and 8 in. of compacted gravel were laid between the brick-earth and the sand, the gravel being consolidated by rollers with rubber tyres. The slabs were laid in rows between 6-in. steel road-forms the distance between which exceeded slightly the width of four slabs. The forms were set to the correct line and level and, with frequent checking by purpose-made straight-edges, the slabs were laid accurately. Spacers were inserted between the slabs to prevent creeping. At the peak period of construction six bricklayers and six labourers were engaged on laying slabs, which were handled by a 15-cwt. mobile crane. Special lifting tongs (*Fig.* 3), which could be withdrawn or re-inserted if necessary without affecting the width of the joint, were used. The paper under the slabs was unrolled immediately prior to laying the slabs to avoid damage by rain and

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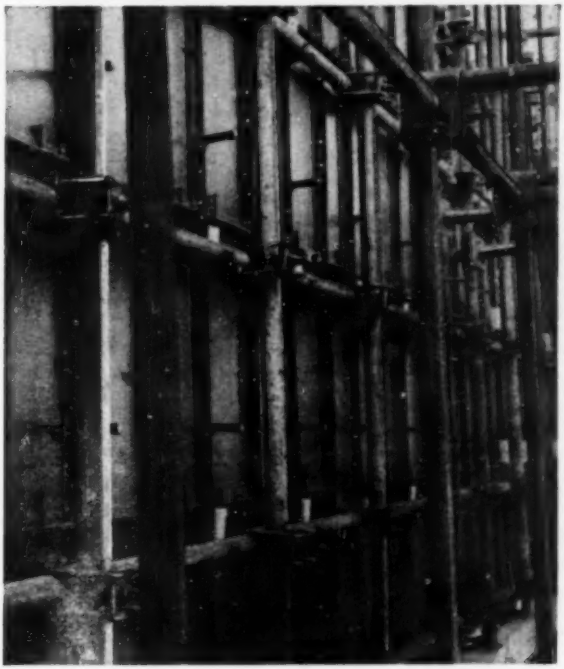

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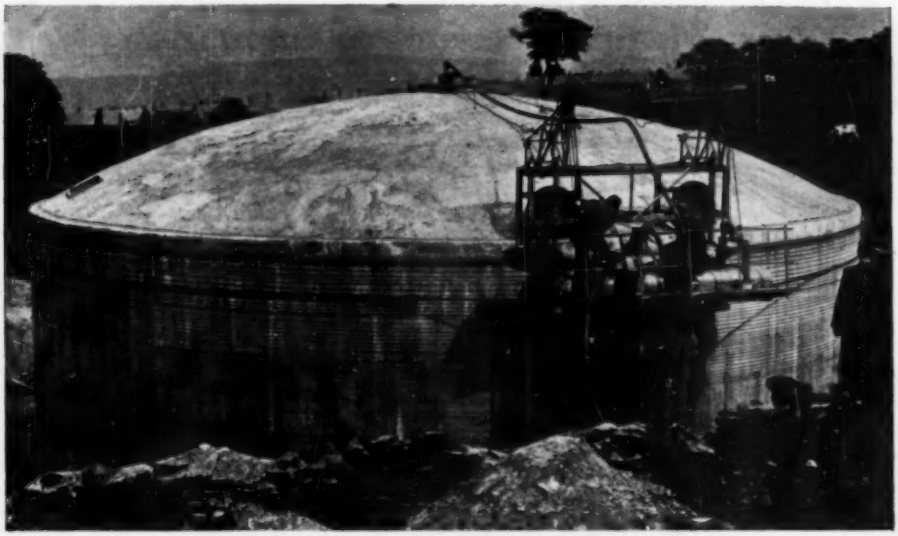

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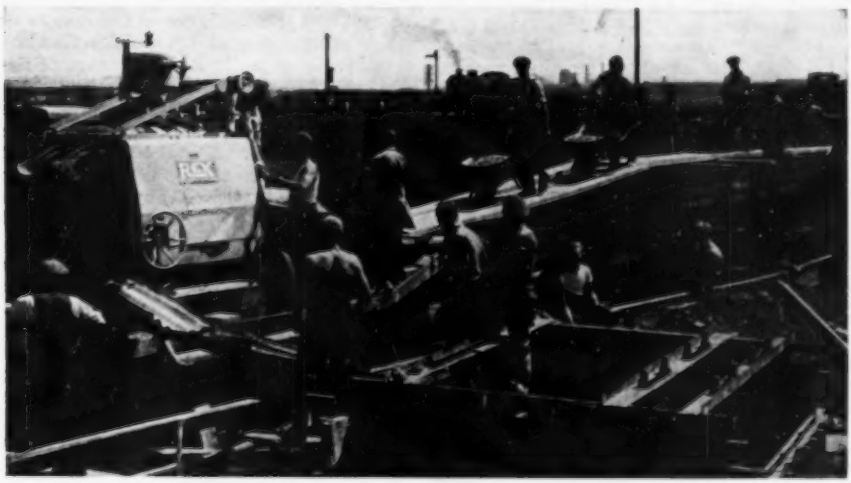
to permit the level of the sand to be adjusted.

The cables were made in a hut on the site in such a position that they could be drawn out of the hut directly into their position in the work. The wire was unwound from drums at one end of the hut and passed through a straightening machine to a bench to which formers were fixed. On completion the cables were painted and wrapped in bituminous paper. The cables, jacks, and cones are the Freyssinet type. After the rollers in the diagonal joints had been fixed, cement mortar was filled to a depth of 2¾ in. in the joints. When this work had advanced sufficiently the placing of the transverse cables commenced. Concurrently with laying the slabs the abutment cables were stretched and anchored. The transverse cables were stretched in sequence. The ends of three cables were anchored and grouted and the protruding wires were burned off to permit the protective concrete edging to be placed.

The taxi-track was designed by the Air Ministry for the Ministry of Civil Aviation. The contractors are Messrs. George Wimpey & Co., Ltd. The precast slabs were made by Girlings Ferro-Concrete Co., Ltd.

Placing Concrete Beside Railway Tracks.



A "Concreting Train."

THE illustration shows a method developed by British Railways for placing concrete in the foundations of a large number of steel structures to carry overhead conductors on lines which are being converted to electric traction. These "concreting trains" comprise a wagon carrying a concrete mixer and others containing cement and aggregates, and a locomotive tender with a capacity of 2500 gallons of water. Planks are laid on the materials wagons to enable the aggregates to be barrowed to the mixer, which discharges along a chute to the foundation. When the materials wagons are empty they are replaced with full ones. Some of the foundations concreted in this way in Lancashire are for steel towers 43 ft. high to carry electrical conductors over thirty tracks.

Grading Concrete Aggregates.

Most research on aggregates for concrete has been concerned with continuous gradings. Except for mortars and for concrete for a few special purposes, gap gradings have not been much investigated, though it has been shown that some gap gradings may be used successfully. Impelled by experiences in districts where aggregates conforming to usual gradings were either absent or very expensive, and satisfied from these experiences that gap gradings could be advantageous, Mr. L. Boyd Mercer (of Melbourne) has made a comparison of concretes made with continuously-graded and gap-graded aggregates. His work has taken several years, and parts of it have been the subject of previous publications. In a recent report * he gives a complete review of his investigations.

The subject is complex, and to investigate fully the effect of forming even a single gap in a particular continuous grading would require experimenting with several mixtures. Mr. Mercer wisely began by restricting his experiments to concretes made with the same cement content (564 lb. per cubic yard), three water-cement ratios (0·5, 0·6, and 0·7, but 0·6 only for the principal series), one maximum size (1½ in.), and one type of aggregate (crushed dolerite of "poor" shape), both fine and coarse being crushed material. The principal series of tests with a water-cement ratio of 0·6 included three continuous gradings each containing 40 per cent. of fine aggregate (passing No. 4 sieve) and 60 per cent. of coarse aggregate. The grading curve of each, when plotted on the usual semi-logarithmic paper, consisted of straight lines with the change of direction on the No. 4 ordinate in one, on the ⅜-in. ordinate in the second, and on the ¾-in. ordinate in the third. The remainder of the series consisted of systematic sets of gap gradings, the first set omitting each single size in succession up to ⅜ in.-¾ in., the next omitting two sizes in various arrangements, the next omitting three sizes; and the final set omitting four sizes in different arrangements. In all these the proportion of fine material (passing No. 4 sieve) was constant at 40 per cent., and the water-cement ratio at 0·6. Concrete specimens were made by hand tamping and by vibration. Workability and the tendency to segregation were assessed; unit weights, crushing strength, modulus of rupture, and permeability were measured, and the general qualities of the concretes were compared.

Apparently the three "straight-line" continuous gradings were chosen more for their simplicity than for the concrete-making properties of the particular gradings. Mr. Mercer admits that they are not the type of continuous gradings that would make the best concrete, and the quality of the concretes made with them are described as "relatively poor" for the first and "relatively fair" for the second and third. Some of the concretes made with gap gradings are described as "relatively good." An advantage of the use of these straight-line gradings for comparison is that a good gap grading is apparent because it makes better concrete than the three continuous gradings, but it has the disadvantage of not showing the relationship between the gap gradings and the continuous grading most suited to the mixture and shape of aggregate. A clearer result would have been given if a continuous grading suitable for making relatively good concrete had been included in the series. Such concrete would probably have been at least as good as that made with the best gap grading, and possibly there would have been less tendency to segregate.

A complete statement of the properties of the aggregates and the numerical results of the tests is given. Equally good descriptions of the qualities of the resulting concrete are also given, except that tendencies to segregation are not clearly stated and are left to be inferred. A notable feature is a series of full-page plates each containing the graph of a grading and the water-cement ratio, and underneath a statement of the quality of the concrete. The value of the water-cement ratio given in plate VII is not in agreement with the value in Table V. A clear statement about the tendency to segregation would have improved the value of each note.

Seven pages are given to conclusions

* "The Law of Grading for Concrete Aggregates." By L. Boyd Mercer. 113 pages, including 61 pages of diagrams, plates, and photographs. Published by Melbourne Technical College Press. No price stated.

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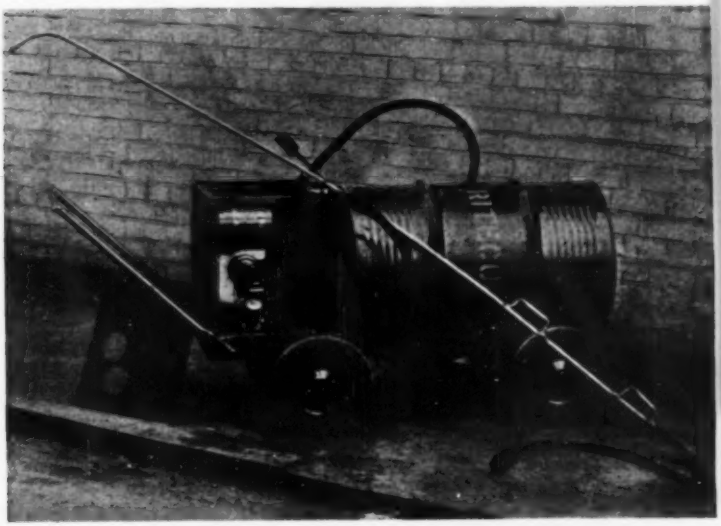

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and suggestions for designing mixtures. Some of these conclusions and recommendations are new. Some are deduced from the experiments and agree with existing knowledge. The following statement may mislead some readers: "Gap Gradings and their Application.—Concrete mixes with aggregates having discontinuous or gap gradings can present superior placeability and behaviour by comparison with those having continuous 'straight-line' gradings for the same water-cement, cement-aggregate, and sand (– No. 4) to stone (+ No. 4) ratios." This is justified by the test results and is quite true, but it can be misleading because the following statement on a later page is separated from it and not emphasised: "The straight-line continuous grading is adopted as presenting the simplest basis for comparison even though selected departures can present superior results."

Similarly, the following suggestion for designing proportions may be indirectly misleading: "The basis for determining the most desirable combination of aggregates containing gaps can be obtained by graphically presenting such combination so that it approximates, with the following limitations, to a continuous grading such as 'straight-line' H, with inflection at $\frac{3}{8}$ in." One tends to assume that the "straight-line" grading H is the best continuous grading, and it is only on turning to plate IX that it is seen that the concrete made with it was only of "fair" quality. These risks of misinterpretation would have been avoided if a continuous grading suitable for making good-quality concrete had been determined, included in the tests, and used as a criterion. It would also have given a clearer comparison of the values of the gradings tested.

However, these are small defects in a publication of great value, and the criticisms are intended to be constructive and not to detract from the value of a fine research. Mr. Mercer's work is worth studying by all interested in the design of concrete mixtures, and especially by those who must use aggregates that cannot be combined so as to produce gradings which agree with those generally recommended.—H. N. WALSH.

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A New Shutter Prop.

Figs. 1 and 2 give details of a steel prop for shuttering made by Ruuk Mulder & Co. in Holland. The wedges through the staggered openings in the four faces of the square upper part of the prop permit vertical adjustment to the nearest millimetre. The wedge bears on the top of a steel scaffold tube which forms the lower part of the prop. The illustrations are taken from a journal entitled "Cement", published in Holland.

Fig. 1.

Fig. 2.

Research on Bamboo Reinforcement in India.

An investigation has been made at the Forest Research Institute, Dehra Dun, India, on methods of waterproofing bamboo with a view to its use as reinforcement in concrete. It was found that bamboo embedded in wet concrete absorbed water up to 300 per cent. of its own weight, and increased considerably in volume. When it was treated with a solution of 40 per cent. resin in alcohol, followed by a coat of white lead, only 1 per cent. of moisture was absorbed in 24 hours; when two coats of white lead were applied the absorption was 0·30 per cent. and the expansion almost negligible.

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SITUATION VACANT. Sales representative required for proprietary method of concrete building construction. Applicants should have had some experience of the building industry. Send full details of previous experience, age, and salary required. TARSLAG LIMITED, Wolverhampton.

SITUATION VACANT. Research assistant required for concrete laboratory. Some experience in reinforced concrete design an advantage. Apply stating experience and qualifications. PROFESSOR A. L. L. BAKER, City & Guilds College, South Kensington, London, S.W.7.

SITUATION VACANT. A British firm of contractors and civil engineers operating in Malaya and Borneo require an assistant engineer for work at their head office, Singapore, connected with public works contracts in Borneo, both civil engineering and building projects, including design work. Write, with full particulars, to Box DE/72, c/o 95 Bishopsgate, London, E.C.2.

SITUATIONS VACANT. General civil and structural designer/draughtsmen are required by Imperial Chemical Industries Limited, General Chemicals Division, to assist in the design of steel and reinforced concrete structures for chemical plants. Location: Runcorn. Salary dependent on age and experience. Apply in writing, quoting E/80, to STAFF MANAGER, IMPERIAL CHEMICAL INDUSTRIES LIMITED, GENERAL CHEMICALS DIVISION, Cunard Building, Liverpool 3.

SITUATIONS VACANT. Structural engineering designer/draughtsmen required in Designs Branch by Air Ministry Works Department. Applicants should have had several years' experience in the design and detailing of reinforced concrete or structural steelwork. The appointments will normally be in London. Salaries are in ranges up to £675 per annum with starting pay dependent upon age, qualifications and experience. Applications, stating age, qualifications, previous appointments (with dates), should be sent to AIR MINISTRY DIRECTORATE GENERAL OF WORKS, C.22 (W.9), Bush House, S.E. Wing, Strand, London, W.C.2, from which address further details may be obtained.

SITUATION VACANT. Prestressed concrete pipe manufacturers require qualified mechanical engineer (A.M.I.Mech.E. or equivalent), preferably with some electrical experience, to supervise the erection of a pipe factory, Liverpool area. Previous experience in charge of small factory installation essential, and knowledge of production planning an advantage. Prospect of management of two or more similar factories. Salary according to qualifications and experience; not less than £750 envisaged. Full particulars to PRELOAD (Gt. Britain), LTD., 5 Stanhope Gate, London, W.1.

SITUATION VACANT. Cost clerk required for precast concrete works, Herts. Must be fully experienced in bonusing, abstracting, and all costing routine. Housing available. Write fully, experience, salary, age, when free, etc. Box 2521, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Structural or civil engineer required by building and civil engineering contractors, HQ Westminster, as assistant to managing director. Age limits 25–40. Engineering degree an advantage, and must have had drawing-office experience in design and detailing of reinforced concrete structures, together with sound practical site training in civil engineering work. Subject to practical proof of ability and satisfactory service, will eventually lead to important executive position. Write giving full details of experience, age, present salary, when available, etc. Box 2522, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. NORTH THAMES GAS BOARD. Junior draughtsmen are required in the chief engineer's department, Westminster. Applicants, who must be fully trained, should have some experience of any of the following: Coal carbonising plant, by-products plant, steel structures, pipe layouts, materials-handling plant, or reinforced concrete. Starting salary, depending on age and qualifications, will be within the range of £415–£555 per annum. The appointment is of a permanent nature, and pension arrangements will be discussed with short-list candidates. Applications, giving age, qualifications, and experience, should be sent to the STAFF CONTROLLER, NORTH THAMES GAS BOARD, 30 Kensington Church Street, London, W.8, quoting reference number 666/7.

SITUATIONS VACANT. Designers, designer-draughtsmen, and detailers required in reinforced concrete construction for offices in Manchester. Progressive positions, with entry into pension scheme after twelve months' service. Commencing salaries according to experience and ability. Address applications, as detailed as possible, in confidence, to "TWISTEEL" REINFORCEMENT, LTD., 7 Oxford Road, Manchester, 1.

FOR SALE.

FOR SALE. Sacks, bags, and curing cloths for sale. You want the best type and quickest delivery. Write JOHN BRAYDON, LTD., 26 The Highway, London, E.1. Telephone: ROYal 1044.

Unset Cement Resistant to Moisture.

It is reported that a new type of cement whose properties are unaffected by prolonged storage in moist conditions, or even by brief contact with water, is being used in Russia. It is stated that when the cement is mixed with aggregate the water-resistant film is removed and the cement sets and hardens in a normal manner.

Emptying Cement Bags.

THE cost of labour in opening cement bags by hand is avoided by a device, described in " Engineering News-Record ", used in the production of 500,000 tons of concrete for a hydro-electric works in Idaho, U.S.A. The bags are taken off the stacks in the cement store and placed by hand at 8 ft. intervals on a belt-conveyor which carries the bags to the top of a short inclined chute. In sliding down the chute the bags pass over two circular saws, which rip them open at a rate of 23 per minute. The cut bags pass on to a vibrating chain where the cement is shaken into a hopper from which it is conveyed to the silos of the batching plant. The empty bags are taken on a belt-conveyor to an incinerator.

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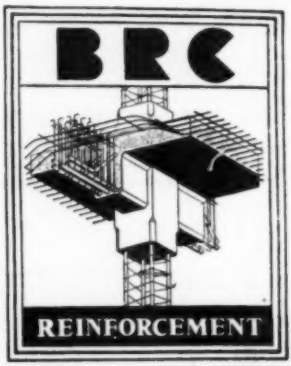
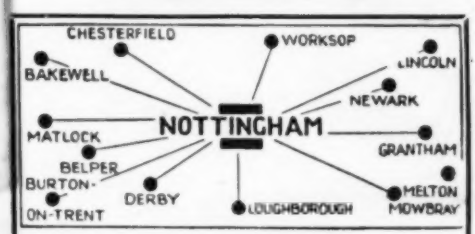
Prestressed Concrete.

A BROCHURE entitled "Prestressed Concrete Manual," in which the Magnel-Blaton system is described and illustrated, is available from Stressed Concrete Design, Ltd., 54 Clapham Common South Side, London, S.W.4.

Change of Name.

The name of Kent & Sussex Contractors, Ltd., has been changed to Rush & Tompkins, Ltd. The Company's business and general organization remain unchanged. The name of the associated company, Rush & Tompkins, Ltd., of Footscray High Street, Sidcup, has been changed to Rush & Tompkins (London), Ltd.





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DATA FOR PRICING REINFORCED CONCRETE.

Materials.

(*Delivered in London area.*)

AGGREGATES (per cu. yd.).—Washed sand, 20*s.* Clean shingle: ¾ in., 17*s.*; ½ in., 19*s.* 6*d.* Thames ballast, 18*s.* 9*d.*

CEMENT (per ton, delivered at Charing Cross).—Portland cement, 6 tons and upwards, 95*s.* 3*d.* 1 ton to 6 tons, 100*s.* 3*d.* Paper bags and non-returnable jute sacks included.

Rapid-hardening Portland, 8*s.* above ordinary Portland.

Aquacrete and 417, 32*s.* 6*d.* above ordinary Portland; paper bags included.

Colorcrete (buff, red, and khaki), in 6-ton loads, 136*s.* 9*d.*; paper bags included.

Snowcrete, £12 12*s.* 9*d.*, inc. paper bags.

"Super-Cement," 32*s.* 6*d.* per ton above ordinary Portland cement; paper bags included.

High-alumina cement, 1 ton and upwards, 255*s.* per ton; paper bags 22*s.* per ton extra.

Snowcem paint, 71*s.* per cwt. inc. containers.

SHUTTERING.—For prices of timber, refer to S.R. & O., 1949, No. 1079 (price 1*s.* 1*d.*) and No. 94 (price 5*d.*) issued by H.M. Stationery Office.

REINFORCEMENT.—Mild steel round bars (per cwt.); ¾ in. to 2¾ in., 35*s.* ⅝ in. to ½ in., 36*s.* ¾ in., 37*s.* ¼ in., 39*s.*

Materials and Labour.

(*Contracts up to £5000. Inc. 10 per cent. profit.*)

PORTLAND CEMENT CONCRETE, 1 : 2 : 4.—Foundations, 3*s.* 8*d.* per cu. ft. Columns, 3*s.* per cu. ft. Beams, 3*s.* per cu. ft. Floor slabs 4 in. thick, 9*s.* per sq. yd.; Do., 5 in., 11*s.* 3*d.*; Do., 6 in., 13*s.* 6*d.*; Do., 7 in., 15*s.* 9*d.* Walls 6 in. thick, 13*s.* 6*d.* per sq. yd. Add for hoisting 3*s.* 6*d.* per cu. yd. above ground floor level. Add for rapid-hardening Portland cement 2*s.* per cu. yd.

REINFORCEMENT.—Mild steel round bars, including cutting, bending, fixing, and wire (per cwt.)—¼ in. to ¾ in., 56*s.* ⅝ in. to ½ in., 50*s.* ¾ in. to 2¾ in., 47*s.*

SHUTTERING AND SUPPORTS.—Walls, 175*s.* per square.

Floors (average 10 ft. high), 180*s.* per square. In small quantities, 2*s.* 3*d.* per sq. ft.

Columns, average 18 in. by 18 in. (per sq. ft.), 2*s.* 3*d.*; in narrow widths, 2*s.* 4*d.*

Beam sides and soffits, average 9 in. by 12 in. (per sq. ft.), 2*s.* 4*d.*; in narrow widths, 2*s.* 5*d.*

Raking, cutting, and waste, 6*d.* per lin. ft.

Labour on splays, 3*d.* per lin. ft.

Small fillets to form chamfers, 6*d.* per lin. ft.

Wages.

The rates of wages on which the above prices are based are: Carpenters and joiners, 3*s.* 3*d.* per hour (carpenters 2*d.* a day tool money); Labourers, 2*s.* 9½*d.*; Men on mixers and hoists, 2*s.* 11½*d.*; Bar-benders, 3*s.*

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THIS WINTER

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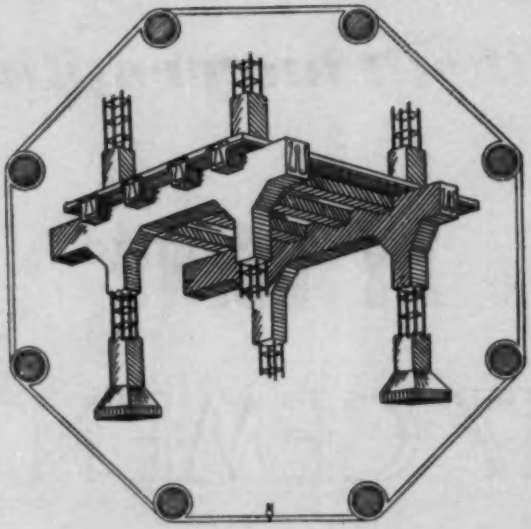

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